# SITE INVESTIGATIONS
# FOR RESIDENTIAL DEVELOPMENT
# ON THE PORT HILLS,
# CHRISTCHURCH

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A thesis
submitted in partial fulfilment
of the requirements for the Degree
of
Master of Science in Engineering Geology
in the
University of Canterbury
by
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University of Canterbury
1989

THESIS with 2 separate items in back pocket



Frontispiece : View North Across Cut and Fill Operations at Westmorland

## ABSTRACT

Three site investigations for residential development on the Port Hills gave a chance to document remedial measures in volcanic bedrock (McCormacks Bay) and cut and fill operations in loess (Westmorland), and to carry out detailed logging and index testing, as well as strength testing in loess (Westmorland and Coleridge Tce).

A design-as-you-go approach was adopted for remedial measures in blast-damaged volcanic bedrock at McCormacks Bay Quarry Subdivision because of potential difficulties in obtaining detailed sub-surface information. Remedial measures included: (a) removal of loose blocks, (b) reinforced concrete buttressing, (c) a gabion basket retaining wall, and (d) a vegetation programme.

Engineering geological mapping and face logging are important for delineating and subdividing rock and soil units, as well as active and inactive areas of erosion and slope instability. Geotechnical testing programmes, remedial measures and earth works should only proceed after completion and interpretation of engineering geological plans, sections and face logs.

Index tests carried out on loess from Westmorland and Coleridge Tce included: (a) grainsize distribution, (b) Atterberg limits, (c) insitu dry density and moisture content, (d) pinhole erosion, and (e) the crumb test for clay dispersion. Grainsize distribution and Atterberg limits are important tests for identifying a material as loess, but show little variation within loess. Dry density, pinhole erosion and detailed field descriptions from a fresh face allow for the division of insitu loess into layers that represent primary airfall and reworked loess, as well as modification by soil/fragipan forming processes.

Total strength parameters ($c,\phi$) were obtained for loess by triaxial testing (UU test) of 35mm diameter tube samples. Maximum strength measured was c=178 kPa, $\phi=30^0$ (W=8.5%), with a minimum of c=0 kPa, $\phi=30^0$ (W=19%).

A comparison of field density tests on loess fill showed that physical tests (tube samples, Balloon densometer, sand replacement) are directly comparable, while results from a nuclear densometer require simple correction factors to be comparable with physical tests.

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## AKNOWLEDGEMENTS.

I wish to aknowledge the following people for help and guidance through this thesis.

Ronald Adlam, David Barrell, David Bell, Sandra Beltman, Terry Burns, Alan Cochran, Stefan Goldwater, Jimi Hendrix, John F. Kennedy, Peter Kingsbury, Maya Malins, Bob McGiffen, John Phillips, Chris Thornley, Dale Waters, Carol Wiltshire.

# CHAPTER 1: INTRODUCTION.

## 1.1 PROJECT BACKGROUND.

Three engineering geological site investigations were carried out as case studies in conjunction with on-going residential development on the Port Hills, Christchurch. The three sites are located on the lower flanks of the Port Hills, immediately SE of Christchurch (Figure 1.1).

McCormacks Bay Quarry Subdivision (Chapter 2) is situated on a disused bedrock quarry (5-50m above MSL) midway between Christchurch and Sumner. Coleridge Tce Loess Bank (Chapter 3) is centered on an east-facing bank (approximately 50m above MSL) along Coleridge Tce, Lyttelton. Westmorland Subdivision Extensions (Chapter 4) occupy a loess covered spur (approximately 100m above MSL) on the SE edge of suburban Christchurch.

These studies utilised the standardised engineering geological assessment approach outlined by Bell & Pettinga,1985 (see Appendix A), and further tested the applicability of the approach. Table 1.1 summarises the Bell approach to urban development in New Zealand. McCormacks Bay investigations and remedial works fit into the scheme at stage F (Table 1.1), and a discussion of problems caused by relatively late engineering geological data input is given in Chapter 2. Westmorland investigations range from stage D to F (Table 1.1). Coleridge Tce investigations lie at stage E, with an engineering geological model and quite comprehensive geotechnical information required by the engineer for finalisation of retaining wall design.

## 1.2 THESIS OBJECTIVES.

The major thesis objective was to carry out case studies on residential development of the Port Hills, and meet the specific objectives for each site studied. Main objectives for each site are :

(a) Document engineering geology and remedial measures, comment on past quarrying methods, and the land subdivision process for McCormacks Bay Quarry;

(b) Log detail of loess bank, characterise materials by index tests and a shear strength testing programme, and produce an engineering geological model for the Coleridge Tce Loess Bank;

(c) Map a corridor for the Penrrudock Rise extension of Westmorland subdivision, monitor fill operations and compare density testing methods, and characterise insitu loess materials by field descriptions and index

Figure 1.1 : Location of Case Study Sites

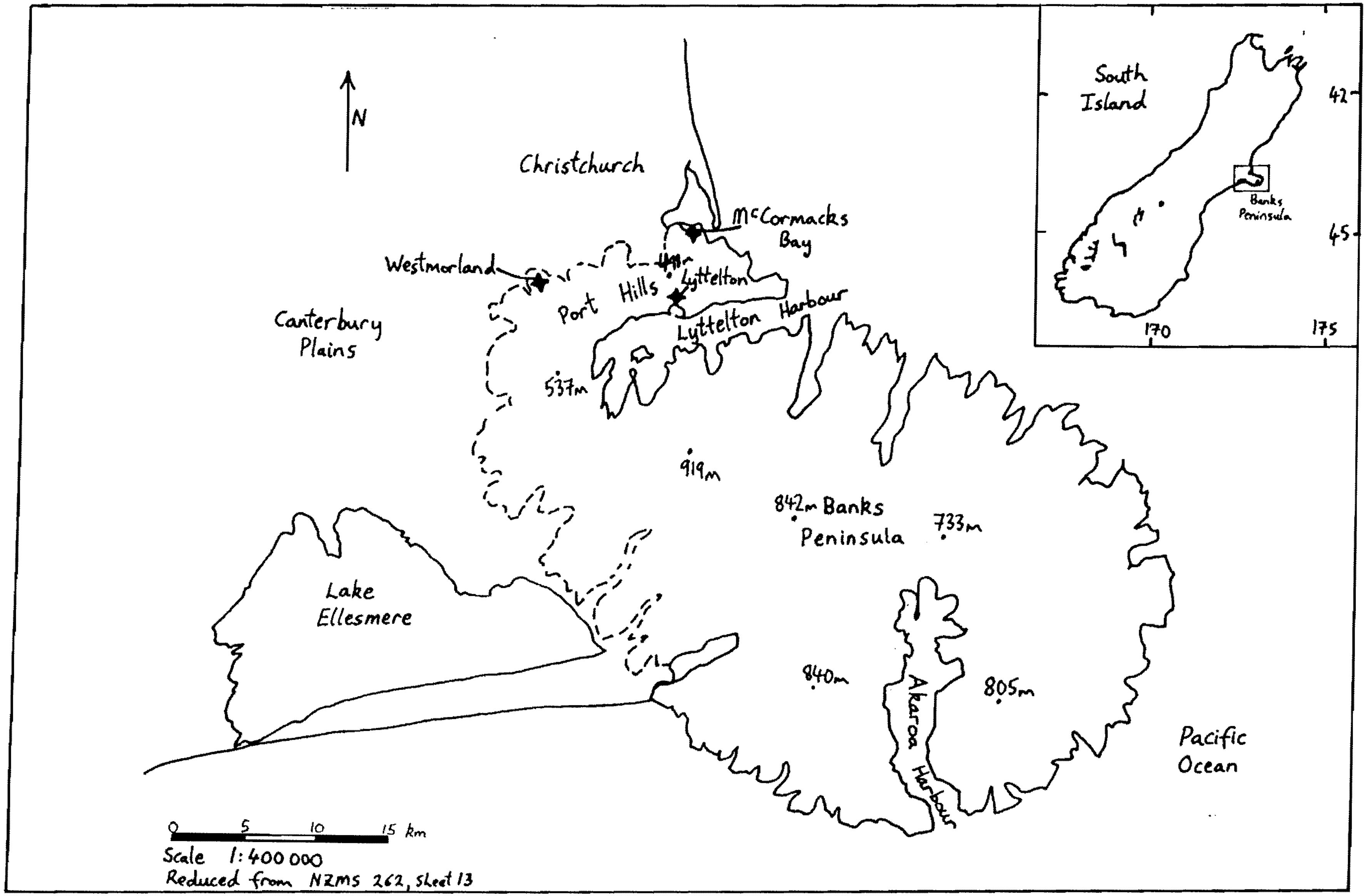


Table 1.1 (after Bell, 1984)

ENGINEERING GEOLOGY DATA INPUT FOR URBAN DEVELOPMENT IN NEW ZEALAND

| Planning Stage[1] | Engineering Geology[2] Investigation Objectives | Typical Map Scales | Geotechnical Data[3] |
|---|---|---|---|
| *A. REGIONAL SCHEME* | 1. Identification of "regional" hazards such as floodplains and "active" fault traces<br>2. Mapping of bedrock and surficial geology | 1:100,000<br>↓<br>1:25,000 | Characterisation of lithologies and identification of "problem" soil types; assessment of resources (e.g. aggregate availability and long-term requirements) |
| *B. DISTRICT SCHEME* | 1. Engineering geological and/or pedological mapping, with limited excavation logging<br>2. Identification and investigation of "local" hazards (e.g. landslides) | 1:10,000<br>↓<br>1:5,000 | Geotechnical characterisation of mapping units as required for land-use zoning decisions; specific evaluation of tectonic and hydrologic hazards |
| *C. SUBDIVISION CONCEPT PLAN* | 1. Engineering geological site mapping and subsurface investigations<br>2. Interpretative risk assessment and/or planning guidelines | 1:2,000<br>↓<br>1:1,000 | Limited testing (e.g. plasticity/grainsize) to indicate general characteristics of site materials; hazard avoidance or mitigation measures |
| *D. SUBDIVISION SCHEME PLANS* | 1. Detailed site investigation of specific areas identified at Concept Plan stage<br>2. Engineering geological mapping and logging to meet any "local" authority requirements | 1:1,000<br>↓<br>1:500 | Additional geotechnical testing to verify design and/or construction feasibility as required; investigation of specific features to facilitate stage E |
| *E. SUBDIVISION DESIGN AND CONSTRUCTION* | 1. Confirmation of mapped geology<br>2. Additional investigation as required | 1:500<br>↓<br>1:50 | Detailed investigations for design of cut and fill batters if required; control of earthworks |
| *F. SECTION DEVELOPMENT AND HOUSE CONSTRUCTION* | Engineering geological investigations only if required (A → E should prevent site "problems") | 1:200<br>↓<br>1:50 | Site specific testing for foundations if required; control of earthworks, drainage, etc |

NOTES: 1) Planning stages follow from existing legislative framework (Table 3).

2) Engineering geology investigation methods include air-photo interpretation and relevant mapping and logging techniques.

3) Geotechnical design investigation requirements may vary considerably within individual urban areas.

tests of cut areas.

## 1.3 GEOLOGICAL HISTORY.

The Port Hills form the NW margin of the Lyttelton sector of Banks Peninsula, a 1200 $km^2$ area comprising the two deeply dissected composite volcanos of Lyttelton and Akaroa (Weaver & Sewell, 1986). A simplified map of bedrock geology is presented in Figure 1.2, with loess outcrop shown in Figure 1.3 (see section 1.3.8). Previous work on geology and engineering geology of Banks Peninsula is reviewed in Appendix A.

### 1.3.1 Pre-Lyttelton Volcano.

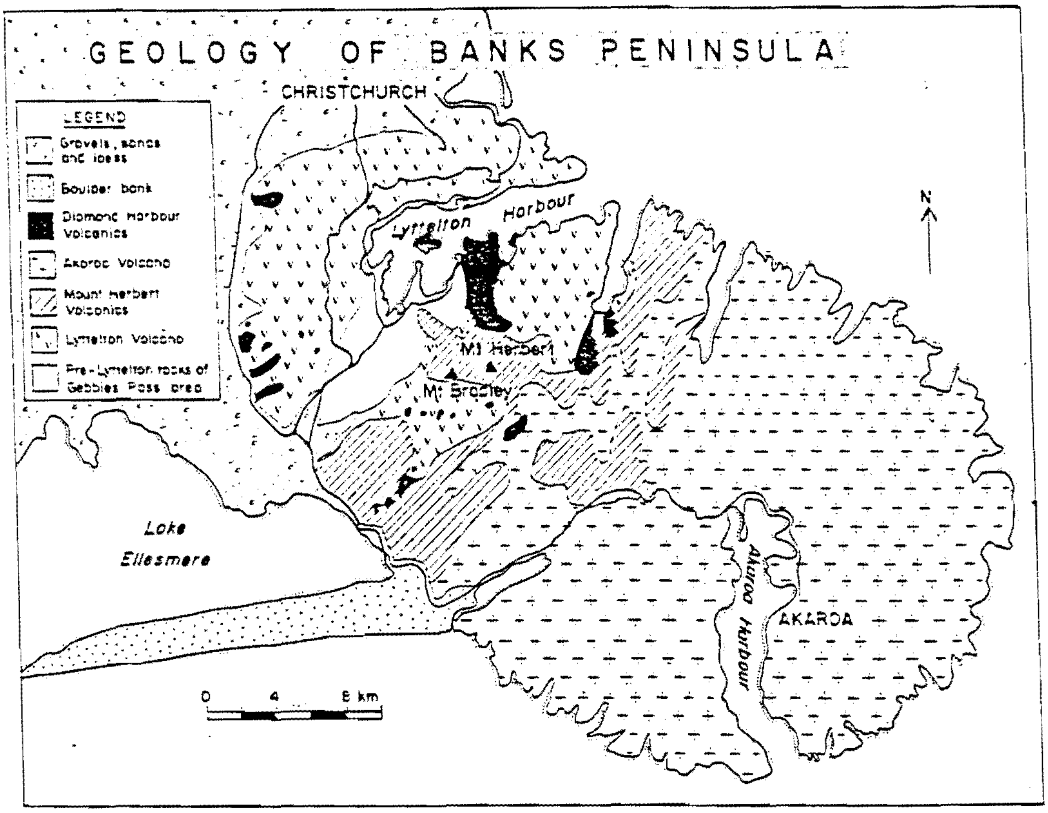
Volcanic activity that built the Lyttelton/Akaroa complex began with the eruption of Governors Bay Volcanics approximately 12 Ma ago (Figure 1.4a) . Governors Bay Volcanics comprise andesite and rhyolite flows and domes erupted onto a basement high of Triassic Torlesse Supergroup rocks overlain by Cretaceous McQueens andesite and Cretacous-Tertiary terrestrial and marine sediments such as Charteris Bay Sandstone (Weaver et al. 1985).

### 1.3.2 Lyttelton Volcano.

The Lyttelton volcano is built of basic and intermediate lavas erupted from centers south and west of Quail Island between 11 and 10 Ma ago (Figure 1.4b) . Minor amounts of airfall deposits and lahars contributed to the stratified nature of the symmetrical dome, which probably rose to 1500 m above present sea level and had a volume of approximately 350 $km^3$ (Sewell, 1988). A radial drainage pattern became established on the dome as evidenced by the restricted occurrence and local thickening of lahars and lava flows. The last phase of Lyttelton activity involved emplacement of a radial swarm of trachyte dykes, with associated domes and flows erupted high on the flanks of the main cone (Weaver & Sewell, 1986).

### 1.3.3 Mt Herbet Volcanics.

Rapid erosion of the "soft center" of Lyttelton volcano led to a breach in the crater wall to the SE. Mt Herbet volcanics were erupted along the SE side of Lyttelton volcano from 9.7 to 8.0 Ma ago (Figure 1.4c,d,e & f), filling the crater breach and blanketing the eroded SE




Simplified geological map of Banks Peninsula showing the distribution of the major volcanic units.

Figure 1.2 (after Weaver et al, 1985)

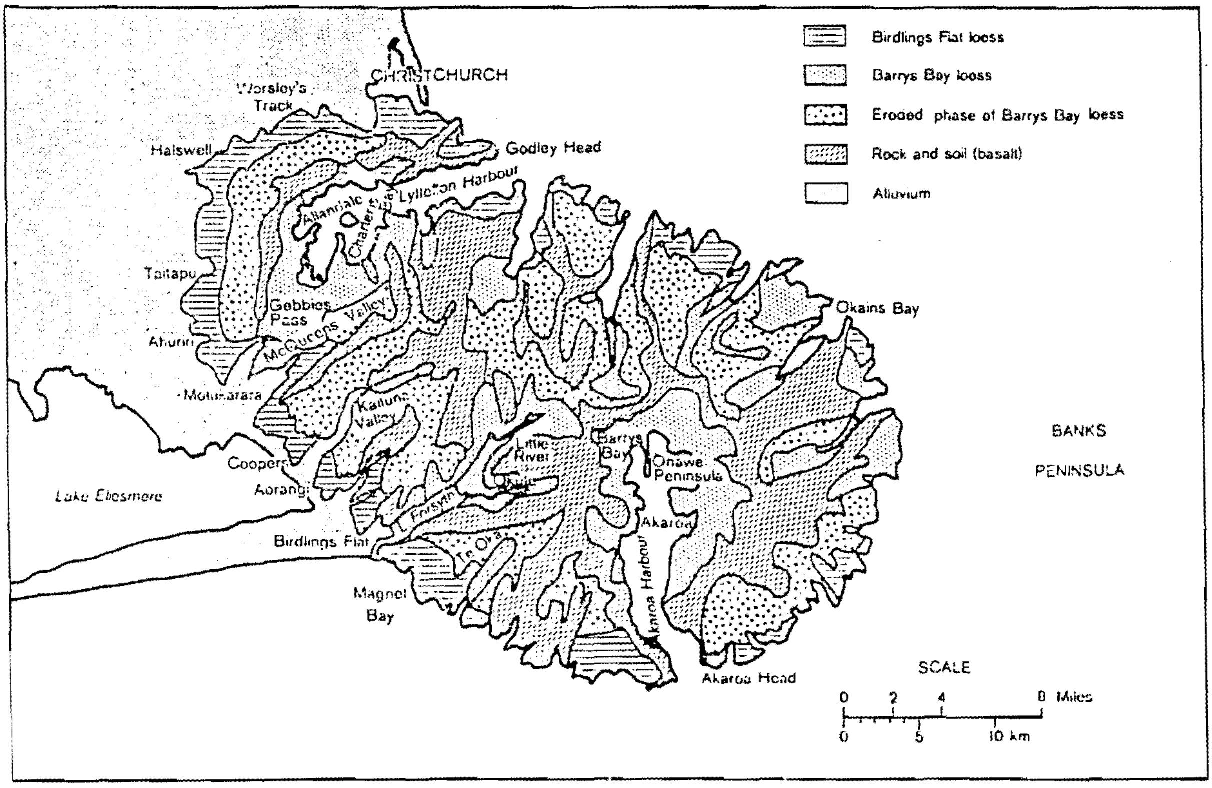
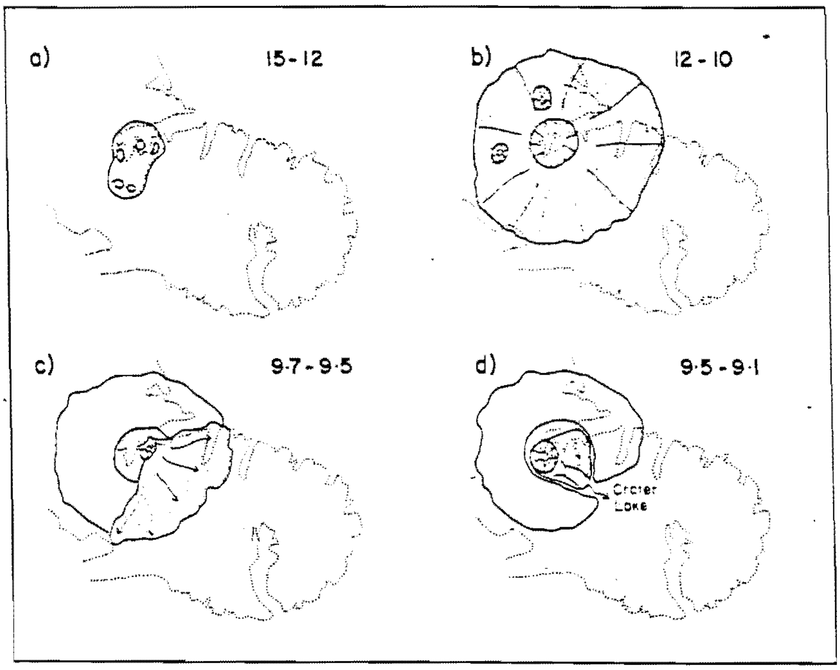
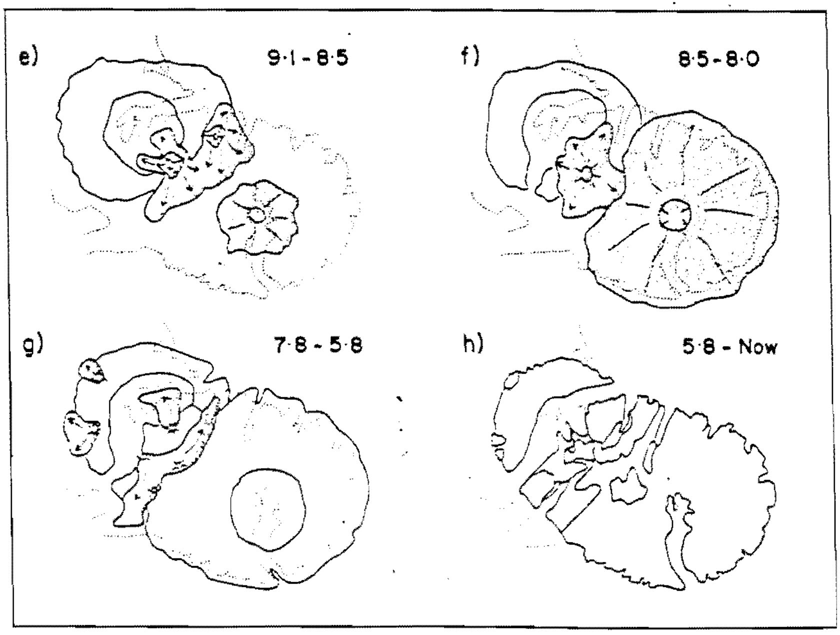



Figure 1.3 Distribution of loess on Banks Peninsula. (after Griffiths, 1973)




Geological evolution of the Banks Peninsula volcanoes.
a) 15-12 million years: Eruption of the Governor's Bay Volcanics from numerous centres at the head of Lyttelton Harbour.
b) 12-10 million years: Construction of the main cone of Lyttelton Volcano.
c) 9.7-9.1 million years: Initial eruptions of the Mt Herbert Volcanics on to the southeastern flanks of the eroded Lyttelton cone, through a breach in the crater wall.
d) 9.5-9.1 million years: Deep excavation of Lyttelton crater by erosion. Formation of a lake on the crater floor and eruption of basalt lavas into the lake water.




e) 9.1-8.5 million years: Further eruptions of Mt Herbert Volcanics from vents close to the crater wall breach and from near Port Levy. Akaroa Volcano becomes active.
f) 8.5-8.0 million years: Construction of the main cone of Akaroa Volcano. Mt Herbert volcanism ceases and a new breach in the Lyttelton crater opens along Gebbies Pass.
g) 7.8-5.8 million years: Eruption of Diamond Harbour Volcanics from numerous vents on the flanks of the eroded Lyttelton crater. Excavation of Akaroa crater and the main channels along which Lyttelton and Akaroa Harbours developed.
h) 5.8-0.0 million years: Development by erosion of the present topography of Banks Peninsula. Gravels of the Canterbury Plains join the island to the mainland.

Figure 1.4 : Geological Evolution of the Banks Peninsula Volcanoes (after Weaver et al, 1985)

flank of Lyttelton volcano (Sewell, 1988).

### 1.3.4 Akaroa Volcano.

Akaroa volcano was built up of basic and intermediate lavas, similar to Lyttelton lavas, erupted from a center at Onawe Peninsula between 9.0 and 8.0 Ma ago (Figure 1.4e,f). Activity at Akaroa was essentially synchronous with the later stages of Mt Herbet volcanics, but approximately 10 times more voluminous (1200 $km^3$ compared with $100km^3$, Sewell 1988).

### 1.3.5 Diamond Harbour Volcanics.

The last stage of volcanic activity at Banks Peninsula involved the eruption of Diamond Harbour volcanics from vents in the deeply eroded Lyttelton crater and around the western flanks of the volcano (Figure 1.4g) . This activity lasted from 7.0 to 5.8 Ma ago (Sewell, 1988) and is responsible for the well developed lava dip slopes that blanket part of the SE wall of Lyttelton crater, as well as the high level intrusions that are worked for roading aggregate at Halswell quarry.

### 1.3.6 Pre-Quaternary Erosion.

Headwards erosion of the radial drainage features proceeded very rapidly in the Lyttelton and Akaroa volcanos. The soft and broken-up material at the core was relatively easy to remove (S. Weaver pers comm 1988). By 8.0 Ma ago erosion had breached Lyttelton crater to the SW through Gebbies pass, and eventually breached it to the NE (approximately 5.8 Ma) developing the present drainage system of Lyttelton Harbour.

Erosion of massive lavas on the lower seaward flanks of the volcanos was much slower, although the present-day valleys incised along the NW edge of the Port Hills would have been basically formed by the onset of glaciation in the Pleistocene.

### 1.3.7 Quaternary Development.

Three major events, that contributed to the present landscape occurred during the Pleistocene. They were: (1) the over-deepening of valley systems on Banks Peninsula during low (glacial) sea level stands, (2) the progradation of alluvial fans to form the Canterbury Plains linking Banks

Peninsula with the mainland, and (3) the blanketing of the Peninsula with airborne silt (loess).

Fluctuations in sea level during the Pleistocene glaciations has resulted in deepening of valleys draining Banks Peninsula during low sea level stands and the infilling of valleys by progradation of sediments during high sea level stands.

Valleys along the NW margin of the Port Hills have been effectively filled with sediment since the prograding alluvial fans of the Waimakariri lapped onto Banks Peninsula in the early Pleistocene (Brown & Wilson, 1988).

Progradation of slope derived, estuarine, coastal, and river sediments (silts, sands, gravels) has continued during the relatively stable sea level stand of the last 6000 years (Gibb, 1986), developing the present sediment filled/drowned valley morphology along the NW margin of Banks Peninsula.

### 1.3.8 Loess Deposition.

Predominantly silt sized sediment (Loess) carried in suspension by the wind was periodically deposited in thick sheets (>1m) over Banks Peninsula. Four periods of loess deposition, separated by periods of relative stability and soil formation, are recognised in the most complete loess sections on Banks Peninsula (Griffiths, 1973). Griffiths also identified 2 major facies of Banks Peninsula loess - Birdlings Flat and Barrys Bay. Birdlings flat loess is coarse and calcareous, while Barrys Bay loess is finer and non-calcareous. Distribution of these facies is shown in Figure 1.3. The differences between facies are thought to relate to distance from source area, local climate conditions (i.e. sub-humid or humid) and vegetation cover (i.e. grassland or forest).

Ives (1973) put forward three prerequisites for loess mobilisation and redeposition as ":(a) availability of material of loess size in source areas, (b) a climatic/vegetation imbalance, to delay re-establishment of new cover once disturbed, and (c) rapid removal of existing cover by some natural agency."

The alluvial fan surfaces to the west of Banks Peninsula are an obvious source of loess material. However similar thicknesses of loess on all sides of Banks Peninsula indicate that the source areas included outwash surfaces exposed on the continental shelf, to the east of the Peninsula, during sea level stands up to 100m below current sea level (Raeside, 1964).

Extensive erosion and reworking during and after deposition of the loess resulted in the removal of loess from higher altitudes (above 270m) and thickening of deposits on the lower slopes (Bell & Trangmar, 1987). Griffiths (1973) showed variations in thickness of insitu loess layers, on a broad spur, that are consistent with colluvial thickening during and immediately after deposition, and before formation of a soil profile.

The surface morphology at the end of the Pleistocene was one of a deeply dissected volcano "softened" by a mantle of loess that was thin and discontinuous at higher altitudes, but thick (up to 20m) and almost totally blanketing on the lower slopes and spurs.

During the relatively stable sea-level conditions of the last 6000 yrs (Gibb, 1986) erosion has continued in the loess deposits, with colluvial deposits thickening on some lower slopes and headward erosion of larger gully systems occurring onto spurs. Major colluvial activity appears to be episodic, limited to wet periods, with triggering of shallow slide/flows and extensive slope wash by high intensity rainstorms (Bell & Trangmar, 1987).

### 1.3.9 Age of Loess.

Radio carbon ($^{14}C$) dates from the Timaru area (Ives, 1973) bracket the deposition of youngest loess at 12000 to 10000 yrs B.P., with cessation of deposition of the penultimate loess around 30000 yrs B.P. These dates, plus supporting dates, such as 17000 yrs B.P. from within the top loess at Barrys Bay (Griffiths, 1973), led to the idea that the top four loess layers in Canterbury were all deposited during the last glaciation. It was thought that loess accumulated during brief periods close to the stadial/interstadial boundaries, with soil profile development during the bulk of the interstadial (Ives, 1973 ; Tonkin et al. 1974).

Goh et al. (1977 & 1978) pre-treated Timaru and Banks Peninsula loess samples to remove carbon contamination and produced significantly older dates. The top of the upper loess at Timaru was confirmed at 10 000 yrs, but the top of the penultimate loess was put back to >49 000 yrs B.P. Similarly the ages for all 3 paleosols identified on Banks Peninsula was 49 000 yrs or greater. Further age control was provided by Kohn (1979) who confirmed the age of a rhyolitic tephra, found in the middle of a 10m thick upper loess layer at Amberley, as 20 000 yrs B.P.

Revisions in dating outlined above indicate that the loess layers below the youngest loess are beyond the range of $^{14}C$ dating. Four major loess layers may be related to older glacial/interglacial cycles rather than shorter stadia type fluctuations. The presence of four loess layers related to four glacial/interglacial cycles would tie in with the recently published stratigraphy for the Christchurch subsurface Quaternary deposits (Brown & Wilson, 1988), which are divided into 4 gravel (glacial) and 4 fine sediment (interglacial) sequences.

## 1.4 ENGINEERING GEOLOGICAL PROPERTIES.

### 1.4.1 Lyttelton Volcanics.

Lyttelton volcanics (see section 1.3.2) are encountered in McCormacks Bay Quarry investigations (Chapter 2). Some characteristics of these rocks, described by Crampton (1985) and relevant to the above investigations, are outlined below.

Crampton (1985, see also Bell & Crampton, 1986) divided Lyttelton volcanics into 3 engineering geology units from mapping for the Magazine Bay tunnel section of the Lyttelton-Woolston LPG pipeline. The style of rock mass defect was determined to be the dominant factor in stability, with material strengths being more than adequate to support the proposed excavations. Table 1.2 sets out the "general mode of occurrence and engineering geological properties of each unit" (Crampton, 1985).

Rock is divided on the basis of lava flow morphology, which is responsible for the style of rock mass defects. The divisions are into: (1) massive lava, (2) rubbly lava, and (3) massive & rubbly lava (see section 2.3.3 and Figure 2.7) . Massive lava occurs in the core of the flow units and is broken by polygonal spaced cooling joints that dip perpendicular to the flow top and base. Rubbly lava generally occupies the boundary areas of flow units. Persistent joints are rare, with the rock having a welded gravel and matrix type structure that is interlocking and self-supporting.

### 1.4.2 Port Hills Loess.

#### 1.5.2.1 Slope Deposits and Slope Processes.

Loess is the dominant slope deposit, especially on the lower slopes of the Port Hills, where residential development is concentrated. However, slope deposits also include volcanic derived material and mixtures of loess and volcanic colluvium.

Bell and Trangmar (1987) produced the following classification of

| ENGINEERING GEOLOGICAL MAPPING UNIT | MODE OF OCCURRENCE | GENERAL ENGINEERING GEOLOGICAL DESCRIPTION | INFERRED BEHAVIOUR DURING TUNNEL EXCAVATION |
|---|---|---|---|
| Basaltic Lavas (LYTTELTON GROUP)<br>(1) Massive Lava | This lava may constitute all, or a section (typically the middle) of a single lava flow. Both section and flow boundaries often difficult to determine. | Slightly to moderately weathered, hard, grey to black massive BASALTIC lava; ranges from slighty to highly vesicular and is porphyritic with plagioclase feldspar phenocrysts up to 10mm dominant.<br>Contains at least 3 joint sets which, are closely spaced, rough to very rough, and infilled with up to 5mm of highly weathered basaltic rock, clay and/or iron oxides. Rare wider spaced joints contain up to 300mm of highly weathered rock. Typical effective block size 100-300mm. | Closely fractured nature and the presence of infilling will probably facilitate joint block gravity falls during and/or after excavation. Rock may be stable where the joints are very rough, tight and contain little or no infilling. Rock break and tunnel shape will be dictated by the orientation of joints; overbreak must be expected. |
| (2) Rubbly Lava | May constitute all, or sections (typically top or bottom) of a single lava flow. Section boundaries with massive lava often difficult to determine. | Slightly to moderately weathered clasts (up to boulder size) of massive BASALTIC lava- in a matrix of; slightly to highly weathered, soft to hard, greyish black and red mixture of silt sand and gravel sized fragments of BASALTIC lava; non-highly vesicular and porphyritic.<br>Predominantly unjointed or contains rare, low persistent (<2m), randomly oriented joints. In some exposures there is a persistent (>5m), moderately dipping (30-50°) joint set striking subparallel to the tunnel line that is rough to very rough and contains up to 100mm of soft ashy or clay infilling, spacing varies from 0.5 to 3m. | Lava will probably break through intact rock material with joints only occasionally dictating rock break and tunnel shape. Minimal overbreak anticipated. Clay infilled joints may dictate tunnel profile and cause some roof and/ or side-wall failures. |
| (3) Massive and Rubbly Lava | May constitute all, or sections of a single lava flow. Boundaries with massive and rubbly lavas often difficult to determine. | A mixed unit containing both rubbly and massive BASALTIC lava.<br>Displays jointing characteristic of both rubbly and massive lava: in rubbly sections joints are rare, of low persistence and are randomly oriented, whereas in massive sections regularly spaced, more persistent joints exist giving rise to an effective block size of between 100 and 300mm. | Will depend entirely on the ratio of rubbly to massive lava: where rubbly lava is dominant, behaviour will be as for rubbly lava above except for localised massive lava sections that may display joint block gravity falls; where massive lava dominates the situation is vice versa. |

Table 1.2 : Engineering Geological Units in Lyttelton Volcanics
(modified after Crampton, 1985)

slope deposits: 1. insitu loess, 2. loess colluvium, 3. mixed deposits of loess and volcanic colluvium, 4. volcanic colluvium, 5. residual volcanic regoliths.

Insitu loess is material that has been in place since cessation of the last major period of loess deposition and is usually capped by an upper fragipan (it may also contain buried fragipans and paleosols).

Loess colluvium is material modified by downslope transport and may be layered or massive, containing upto 10% volcanic derived colluvium. It occurs most commonly burying insitu loess on footslope and lower backslope areas. Many erosion problems in Port Hills loess have occurred in younger loess colluvium deposits that are less compact, have lower density and higher permeability than the bulk of Port Hills loess (Bell & Trangmar, 1987).

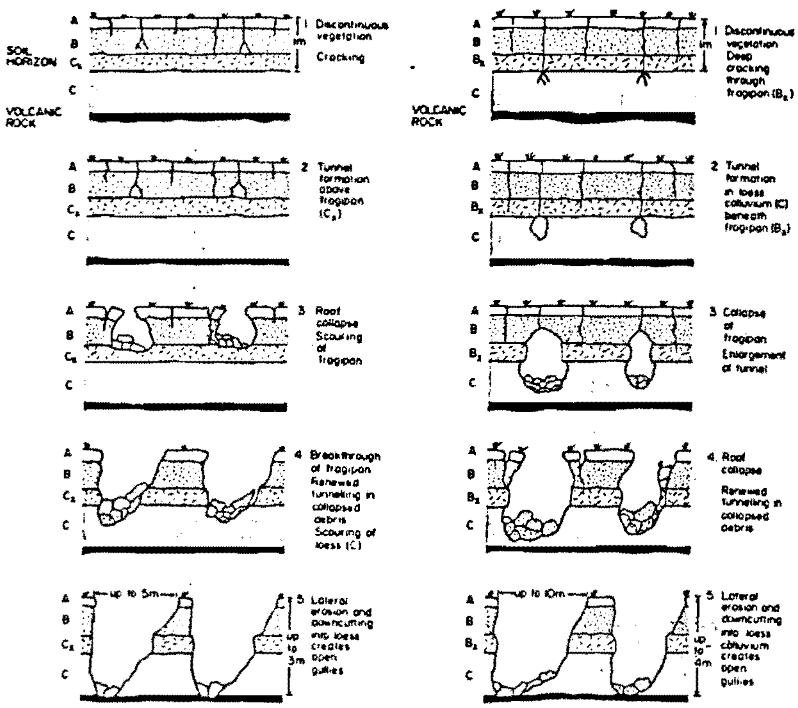
Volcanic colluvium and residual regoliths consist of weathered gravel to boulder size volcanic fragments in a sandy clay matrix of highly weathered volcanics. They occur on, and downslope of rock outcrops and grade through mixed colluvium to pure loess. Mixed colluvium is simply material containing significant proportions (>10%) of volcanic and loess derived constituents, and forms a continuum between volcanic colluvium and loess colluvium.

The slope processes recognised by Bell & Trangmar (1987) are: 1. soil creep, 2. mass movement (slide-avalanche-flow complexes), 3. rock and debris fall, 4. tunnel gullying (subsurface erosion), 5. sheet and rill erosion, 6. wind erosion.

Figure 1.5 gives models for the development of shallow and deep tunnel gullies, involving water penetration along shrinkage cracks and control of cavity development by erosion resistant fragipan layers. Influence of the distribution of colluvial materials on mass movement and tunnel gullying is shown in Figure 1.6. The highest incidence of slide/flows is in the vicinity of mixed colluvium/loess colluvium/bedrock contacts.

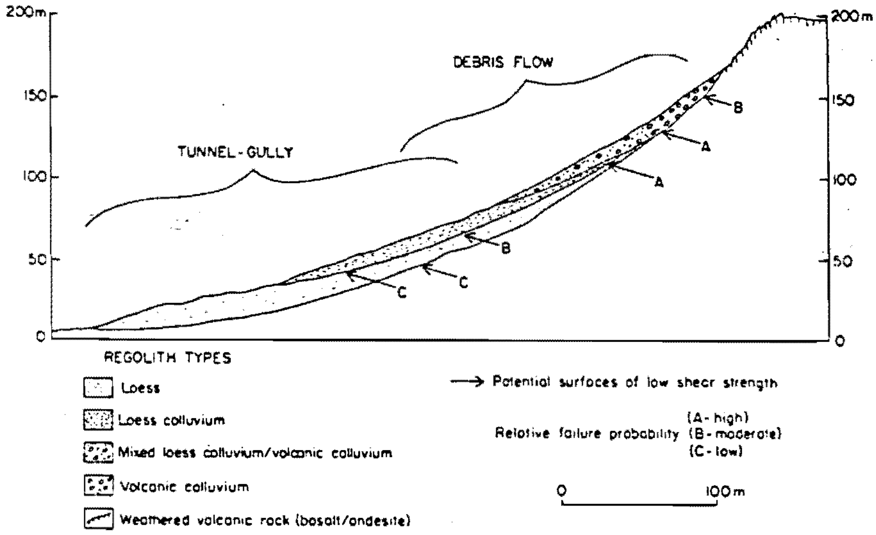
Relationships between slope deposits and processes is shown in Table 1.3. Figure 1.7 illustrates the distribution of deposits and processes across a N-S trending ridge. Insitu loess is preserved on the summit and footslopes, with loess colluvium and mixed colluvium occurring on the backslopes and footslopes.

Tunnel gullies and sheet erosion (slope wash) are the dominant processes on the west facing slopes, while mass movement is dominant on east facing slopes. This variation is attributed to different soil moisture regimes related to slope aspect, i.e. greater seasonal drying and cracking of soil, conducive to tunnel gully development, occurs on W-NW



Models for the formation of A. shallow and B. deep tunnel-gully systems in in situ loess and loess-colluvium regoliths.

Figure 1.5 (after Bell and Trangmar, 1987)



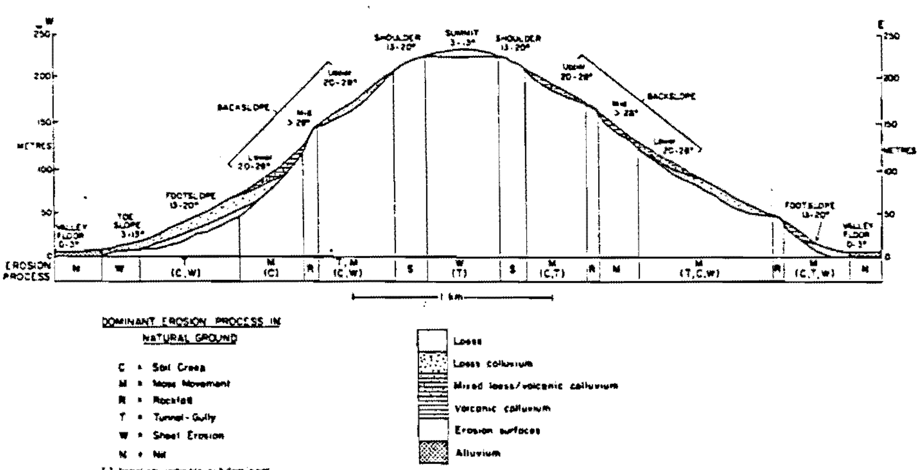

Slope profile in Lyttelton Borough showing relationships between regolith types and erosion processes.

Figure 1.6 (after Bell and Trangmar, 1987)

| Erosion type | Regolith type | Regolith properties influencing erosion | Other factors |
|---|---|---|---|
| TUNNEL-GULLY | Loess, loess-colluvium | 1. Low intergranular cohesion<br>2. Rapid slaking and dispersion when wetted<br>3. Low permeability layers (e.g. fragipan) above which water moves downslope forming tunnels.<br>4. Susceptibility to scouring by moving water<br>5. Cracking in summer due to shrinkage | 1. dry aspects<br>2. seasonal wetting and drying<br>3. discontinuous vegetation cover in summer<br>4. slopes 3-35°<br>5. inefficient stormwater disposal |
| MASS MOVEMENTS (SLIDE-AVALANCHE-FLOW) | Loess-colluvium, mixed colluvium | 1. Impeded internal drainage<br>2. Saturation of material above low permeability layers<br>3. Heterogeneous composition causing variable strength<br>4. Low shear strength when saturated<br>5. Potential failure surface along fragipan, layers of colluvium, colluvium/bedrock or colluvium/loess contact | 1. strong gravitational forces on slopes 18-35°<br>2. shady aspects or gullies<br>3. seepage zones<br>4. cut and fill excavation<br>5. inefficient stormwater disposal |
| | Volcanic-colluvium | 1. Shallow regolith with low moisture storage<br>2. Potential failure surface at colluvium/bedrock contact<br>3. As in 3 for loess-colluvium<br>4. Saturation of material over bedrock contact | 1. slope >26°<br>2. seepage zones |
| SOIL CREEP | Loess, loess-colluvium, mixed colluvium | 1. Impeded internal drainage<br>2. Plastic behaviour of saturated loess under gravitational stresses<br>3. Potential failure surfaces along fragipan, layers of colluvium colluvium/bedrock, or colluvium/loess contact | 1. seasonal and diurnal wetting and drying of soil aggregates<br>2. seasonal and diurnal temperature changes<br>3. stock trampling<br>4. cut and fill excavation<br>5. strong gravitational forces on slopes 7-36°<br>6. in gullies with slow site drainage |
| | Volcanic | 1. Potential failure surface along colluvium/bedrock contact | |
| SHEET AND RILL EROSION | All regolith types | 1. Friable, weakly structured topsoils<br>2. Desiccation of topsoils in summer<br>3. Dispersive soils where loess content high<br>4. Thin, slowly permeable crust formed on soil surface by sheet wash reduces infiltration and increases runoff | 1. summer moisture deficiency causes topsoil desiccation<br>2. discontinuous vegetation cover<br>3. rapid runoff following heavy rainfall, especially from shallow regoliths<br>4. increase in runoff velocity as slope angle increases (slopes >7°)<br>5. severe sheet erosion leads to rilling |

Summary of Relationships between regolith properties and erosion type.

Table 1.3 (after Bell and Trangmar, 1987)




Generalised ridge cross-section showing relationships between landforms, slope, regolith and erosion on the Port Hills

Figure 1.7 (after Bell and Trangmar, 1987) facing slopes (Bell & Trangmar, 1987). Higher year-round soil moisture contents on east facing slopes helps promote soil creep and shallow mass movements (see Table 1.3).

#### 1.4.2.2 Geotechnical Properties.

Geotechnical properties for Banks Peninsula loess have been summarised by Yetton (1986), and include: grainsize, Atterberg limits, dry density, erodibility, dispersion, porosity, void ratio, permeability, shear strength, linear shrinkage, compression index, chemical properties (Ph, soluble salt conc., exchangable sodium percent), and field geophysical parameters (seismic velocity, resistivity, conductivity).

Since 1986 the work of Glassy (1986), Tehrani (1988) and this study have added to the data base. A modified version of Yetton's table is given in Table 1.4. Grainsize is very consistent, as expected from an aeolian sourced material. The silt fraction is commonly >80%, with clay (<0.002mm) ranging from around 10-20% by weight. Similarly, Atterberg limits show very little variation and have low plasticity indices indicating low clay content and insignificant amounts of swelling clays. Insitu dry density has a range of $0.5t/m^3$, with a mean dry density around $1.6t/m^3$.

Clay %, clay mineralogy and soil density effect the major control over the remaining geotechnical properties, e.g. erodibility, dispersion, seismic velocity and chemical properties. Field moisture content is the other main factor influencing geotechnical properties. Shear strength and erodibility, which are the dominant properties controlling slope processes, vary considerably with moisture content (see section 1.4.2.1).

### 1.5 THESIS METHODOLOGY AND ORGANISATION.

Field and laboratory methods used in this study were restricted to simple and relatively fast techniques applicable to normal site investigations, or aimed at specific problems. The principle was to only use methods available during routine commercial investigations, although some methods (e.g. grainsize determination and triaxial testing) were applied much more rigourously than costs would normally allow.

Field methods include: engineering geological mapping and face logging, seismic refraction, back hoe channel logging, nuclear densometer, and physical density tests (i.e. Balloon densometer and sand replacement). Methods are outlined in Appendix 1. Mapping was carried out on existing topographic plans and plans drawn from stadia surveys completed for this

| Parameter | Yetton (1986), including previous work | | | Glassey (1986) 1 bulk sample | | | Tehrani (1988) 1 bulk sample | | | This study (30 samples) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Grainsize (%) | clay 11-25 | silt 65-80 | sand 10 | clay 21 | silt 65 | sand 14 | clay 14 | silt 76 | sand 10 | clay 8-25 | silt 60-80 | sand 10 |
| Atterberg Limits (%) | WL 18-33 | WP NP or 17-22 | | WL 24 | WP 16 | | WL 24 | WP 17 | | WL 21-27 | WP NP or 16-20 | |
| Dry Density (kg/m³) Insitu | 1390 - 1880 | | | 1720 | | | 1550 | | | 1400 - 1900 | | |
| Dry Density (kg/m³) Recompacted | 1800 - 1900 | | | 1850 | | | 1840 | | | - | | |
| Insitu Moisture (W %) | - | | | 7 | | | 7 | | | 4.5 - 21 | | |
| Pinhole Erosion | E50 to E>1000 | | | E180 | | | E50 | | | E50 to E>1000 | | |
| Dispersion (Crumb test) | class 1 to 4 | | | class 4 | | | class 3 | | | class 1 to 4 | | |
| Void Ratio (e) | 0.6 to 0.7 | | | - | | | - | | | 0.55 - 0.75 | | |
| Porosity (%) | 30 to 40% | | | - | | | - | | | - | | |
| Permeability Insitu | around $10^{-7}$ m/s | | | - | | | - | | | - | | |
| Permeability Recompacted | $10^{-8}$ to $10^{-9}$ m/s | | | - | | | - | | | - | | |
| Shear Strength cohesion (c) | 85 to 112 kPa | | | - | | | 30 kPa | | | 0 to 180 kPa | | |
| Shear Strength friction angle (ø) | 30 to 32° | | | - | | | 30-39° | | | 29 to 34° | | |
| Compression Index | Cc 0.17 % | | | - | | | - | | | - | | |
| Ph | 5 to 7 | | | - | | | 7.6 | | | - | | |
| Soluble Salt | 1 to 60 meq/L | | | - | | | 0.36 meq/L | | | - | | |
| Exchangeable Na% | 0.9 to 41 at depth | | | - | | | 0.66 | | | - | | |
| Seismic Velocity | 250 to 400 m/s | | | - | | | - | | | 200 to 700 m/s | | |

Table 1.4 : Geotechnical Properties of Port Hills Loess (modified after Yetton, 1986)

study. Seismic refraction surveys, using a single channel signal enhancement seismograph, were used to determine depths of fill material and depth to volcanic bedrock. Backhoe pits and cuts allowed detailed logging of the soil profile and checks on depth to bedrock predictions. The field density tests determined quality of loess fill after placement.

Laboratory methods include: grainsize analysis by sieve and hydrometer, Atterberg limit determination, pinhole erosion, crumb test for clay dispersion, insitu dry density and moisture content from drive tube samples, triaxial testing for shear strength parameters. The methods used and standards adopted are outlined in Appendix 2.

This thesis comprises 5 chapters. Chapter 1 introduces the area and summarises the geological history and engineering geology. Chapters 2,3,4 deal with specific site investigations, McCormacks Bay Quarry, Coleridge Tce, and Westmorland Subdivision respectively. Chapter 5 contains a summary and conclusions. Much of the methods and results data are contained in Appendices 1 to 7.

# CHAPTER 2 : McCORMACKS BAY QUARRY SUBDIVISION.

## 2.1 INTRODUCTION.

Chapter 2 incorporates site investigations and remedial measures undertaken on two sites forming part of a disused quarry at McCormacks Bay, during 1986 and 1987. A third part of the quarry had been investigated for fill and retaining wall stability in 1985 (section 2.4.1).

The two sites considered here are: (a) Balmoral Partnership, comprising six residential lots off McCormacks Bay Road, and (b) the Wilson property, shown as lot 24 Glenstrae Ave (Figure 2.1).

An outline of the quarry history has been determined from "The Port Hills Of Christchurch." (Ogilvie, 1978), an air photo review and by accounts of local residents. In summary; (1) quarrying along the eastern flank of McCormacks Bay began in the early 20th century when the Tramway company required rock fill to build a causeway spanning the mouth of the bay. Initially small pockets of massive lava were worked from outcrops high on the side of the hill, the rock was transported by chutes to a crushing plant, and then to hoopers at the base of the hill.

(2) Operation of the quarry was transferred from the Tramways Board to the Christchurch City Council during the 1930s, and production from the quarry was used by depression workman for widening of the causeway to accommodate road traffic, as well as for building seawalls at Sumner and along parts of the Estuary foreshore.

(3) Development of the quarry continued into the 1960s, with production used as rock fill and rip-rap for protection of seawalls and stopbanks. Records of areas worked and quarrying practices during the final years are not readily accessible at the Christchurch City Council, and it is possible that formal records do not exist (Mr B. Bluck, pers comm). However, it can be deduced that the minimum age of the final batters in the area of concern is at least 20 years, with a maximum age of about 50 years. Mapping/logging of excavations for remedial measures led to an interpretation of quarry practices employed in developing these batter slopes (section 2.6.2).

## 2.2 SITE DESCRIPTION.

McCormacks Bay Quarry is situated in Lyttelton Group volcanics (Mt Pleasent Formation, Weaver, 1988) which is comprised of basaltic lavas and volcanic derived clastic deposits (e.g. lahars, tuffs, colluvium). The final form of the main quarry development is three benches separated by steep (>30°) batter slopes. Slopes are covered with a variable thickness of loose debris, and vegetated by self-sown grasses, broom, and pine trees. Quarry shape is irregular and reflects the distribution of 'massive lava' (section 2.3.2), which was the most sought after rock because of the potentially large block size and its greater durability and strength compared to rubbly lava and pyroclastic rock types.

The lower bench is the largest and has been subdivided into 6 lots (Balmoral Partnership).Blocky fill brings the lower bench level up to 5m above McCormacks Bay road. A 20m high batter slope, incorporated in the lot titles, backs onto the lower bench and forms the front edge of the middle bench. The Wilson property (lot 24) occupies the middle bench and 10m batter slope behind, with access from Glenstrae Ave. Lot 22 includes the filled bench and massive lava scarp off the end of the 2nd hairpin bend on Glenstrae Ave (Figure 2.1 and 2.2).

## 2.3 NATURE OF MATERIALS.

### 2.3.1 Introduction.

Bedrock and surficial deposits across the quarry area have been divided into seven engineering geological units for the purposes of plan and section compilation. The units were defined on the following basis : (a) readily distinguishable in the field, (b) large enough to map at 1:200 scale, and (c) differences in material and mass characteristics (as well as location of units) gives potentially different responses to extreme conditions such as intense rainfall or earthquake loading (i.e. their engineering geological or geotechnical properties differ). Bedrock units are basically defined on textural differences controlled by mode of emplacement and degree of weathering. Engineering geological descriptions are given below (section 2.3.2), with discussion on occurrence and behaviour in section 2.3.3 and 2.6.1.

### 2.3.2 Descriptions.

Field descriptions follow the scheme of Bell & Pettinga (1984).

(a)'Rubbly lava':

Moderately weathered, moderately strong to moderately weak, dark purple-brown and red-brown, massive, cobble GRAVEL with some clay and

Figure 2.1 : Location of Major Lot Boundaries on McCormacks Bay Quarry

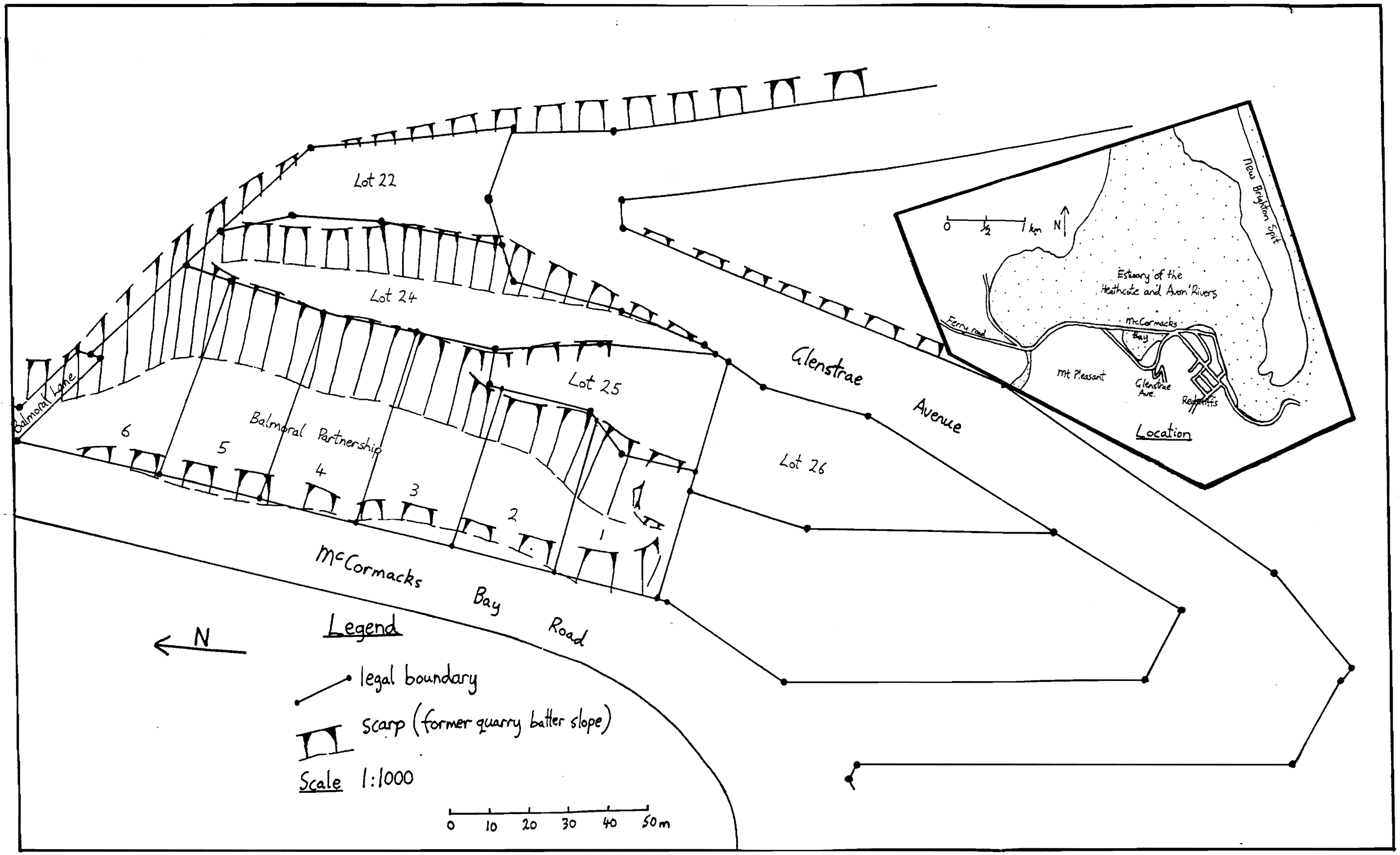



Figure 2.2 : McCormacks Bay Quarry Subdivision. Note retaining wall and buttresses on Lot 24, fill on Lot 22 and crane working on Lot 6 (Balmoral Partnership)

Figure 2.3 : Rubbly Lava with open fractures (arrowed) on north end of Lot 24

silt. Cobble clasts: moderately to highly weathered, weak to moderately strong 'massive' and 'vesicular lava' blocks. Matrix: friable to welded ash-like material. Rockmass: generally massive, occasional persistent, but highly irregular and rough open fracture (Figure 2.3).

(b) 'Massive Lava':

Slightly weathered to unweathered, very strong, dark grey, fine grained olivine BASALT, some plagioclase, olivine and amphibole phenocrysts upto 1cm long. Rockmass: metre spaced, dry, slightly open, curved, subvertical cooling joints generally grouped into 2 sets approximately $90^0$ apart. Cooling joints may be opened, and intact material heavily shattered by blasting (Figures 2.4 and 2.5).

(c) 'Vesicular lava':

Slightly to moderately weathered, strong to moderately weak, dark to light grey, phenocryst and vesicle rich olivine BASALT. Plagioclase and olivine phenocrysts highly weathered, some 'vesicles' are crystal moulds left by weathering of phenocrysts. ('Vesicular lava with clay pods': highly weathered 'vesicular lava' with pods of red/brown plastic clay upto 10cm diameter and irregular zones or blocks of slightly weathered material. Strength ranges from strong to moderately weak in hand speciman. Note this material was moist when freshly excavated.) Rockmass: generally massive, occasional persistent, irregular and rough open fractures - appear to be blast induced. See figure 2.5.

(d) 'Ash/Lahar':

Slightly to moderately weathered, compact, pink/red/grey/brown, layered and massive, SILTS, SANDS and matrix supported GRAVELS. Sands and silts are layered lithic and crystal tuffs. Gravels are massive debris flows or lahars containing clasts of mixed volcanic lithologies in an ash matrix. Rockmass: generally massive with rare, moderately inclined wavy fractures observed in road cuts outside limits of field area.

(e) 'Quarry floor fill':

Variably graded, angular, compact volcanic GRAVEL consisting of fresh to highly weathered massive and rubbly lava blocks, with fines of loess and volcanic ash (sand to clay size). Clay fines plastic. Fines eroded from around some large gravel clasts. Grading of fill, and gravel/fines ratio vary because of poor mixing (Figure 2.6).

(f) 'Blocky Fill':

Angular volcanic GRAVEL with up to boulder size blocks and predominantly loess fines. Similar to 'Quarry floor fill', but dumped onto batter slopes and uncompacted. Significant areas of 'Blocky Fill' occur on lots 1 and 2, Balmoral Partnership.

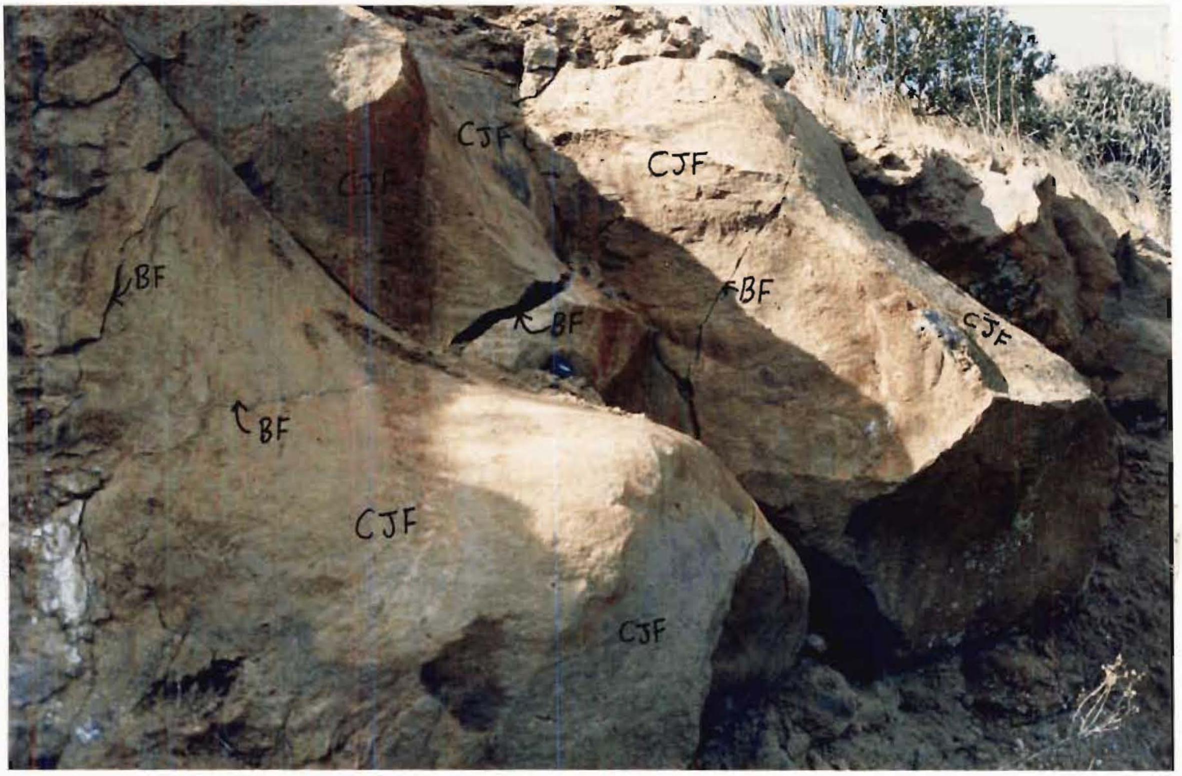



Figure 2.4 : Massive Lava on Lot 24 showing well developed cooling joints and blast fractures. CJF= cooling joint face
BF= blast fracture


Figure 2.5 : Bench Preparation for Gabion Wall. Vesicular lava, vesic. with clay pods, blast fractured massive lava and colluvial material are indicated

Figure 2.6 : Quarry Floor Fill (Balmoral Partnership). Note variety of lava lithologies

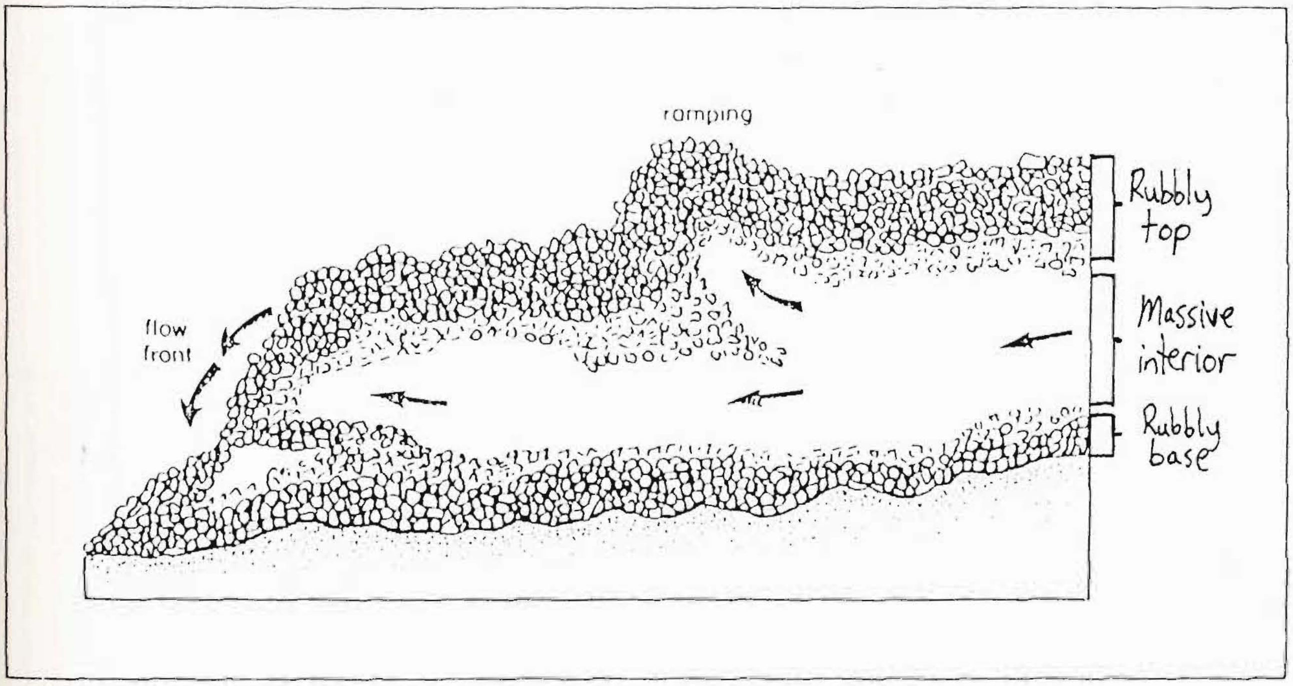



Figure 2.7 : Long Section through a Basalt Lava Flow. The top and bottom surfaces and flow front break up in response to movement within the flow. On cooling the interior becomes solidified massive mava sandwhiched between layers of rubbly lava (modified after Weaver et al 1985)

(g) Colluvium :

Mixture of blast fractured 'massive lava','rubbly lava', volcanic ash loess and topsoil forming a loose sandy GRAVEL with some silt. Where material has been dumped episodically, or slid down slope it may have well defined layering parallel to the slope (i.e. approximately $30^0$ dip).

### 2.3.3 Bedrock Geology.

Bedrock units can be divided into lava flows, debris flows and airfall deposits, which occur in more or less planer layers parallel to the flank slopes of the volcano. At the 1:200 scale used for mapping across this site the thickness of layers varies considerably (both in long and cross section), because of local topographic influence on flows. A lava flow unit can consist of 'massive', 'vesicular' and 'rubbly' lava (see section 2.3.2 for descriptions). 'Massive lava' is slowly cooled material at the core of the flow. 'Vesicular lava' is phenocryst and gas rich (produces vesicles), and may be a product of flow segregation from the massive material. 'Rubbly lava' is the boundary material above and below the flow and is generally broken up and oxidised. Compare this division with that of Crampton, 1986 (Table 1.2 ) who uses only rubbly and massive lava. The further division of 'vesicular lava' is possible at McCormacks Bay because of mappable outcrops of this material, although other sites may require modification or addition of engineering geological units depending on local bedrock details.

Relative thickness and position of the lava flow units varies because of ponding in pre-existing gullies, ramping and mixing by turbulence in the flow. Figure 2.7 is a diagrammatic cross-section of a typical lava flow showing thickening and incorporation of rubbly material within the massive core by the process of ramping. Compare this model with the thickness variations of lava types across the batter slope of lot 24, especially the tongue of 'rubbly lava' protruding into 'massive lava' at the north end (figure 2.8), and the lot 4-6 area of Balmoral Partnership (figure 2.9). On both faces flows have thick 'vesicular' and 'rubbly' sections to the south, with 'massive lava' thickening towards the north while 'vesicular/rubbly lava' thins. Note that the actual lava flow direction was towards the northwest, while quarry batter slopes are aligned north-south giving an oblique flow section rather than a true long section. Ponding of lava in gullies at the base of the volcano's slope (i.e. close to sea level) has resulted in the very thick (>10m) 'massive lava' units worked at the north end of McCormacks Bay Quarry.

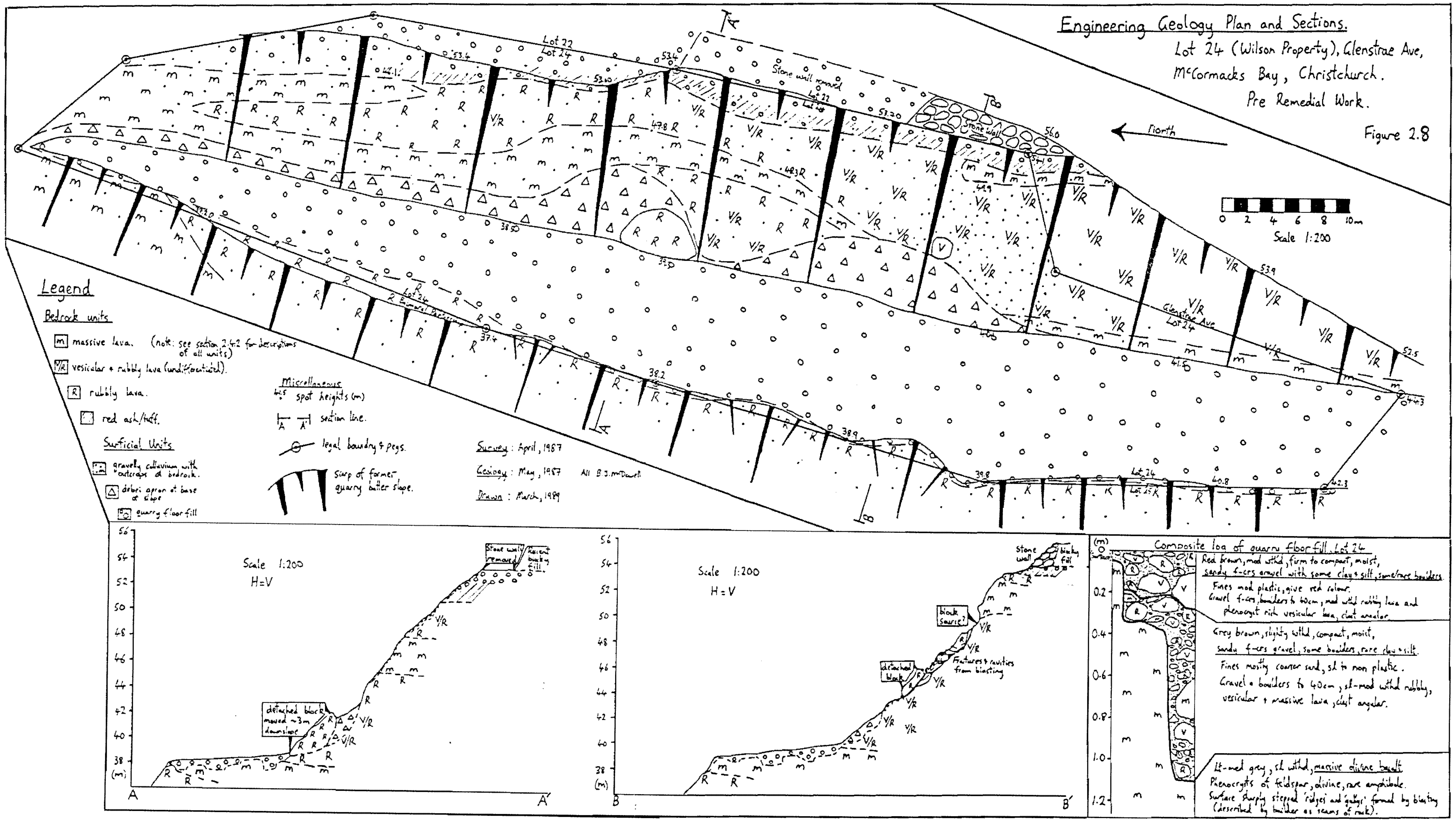
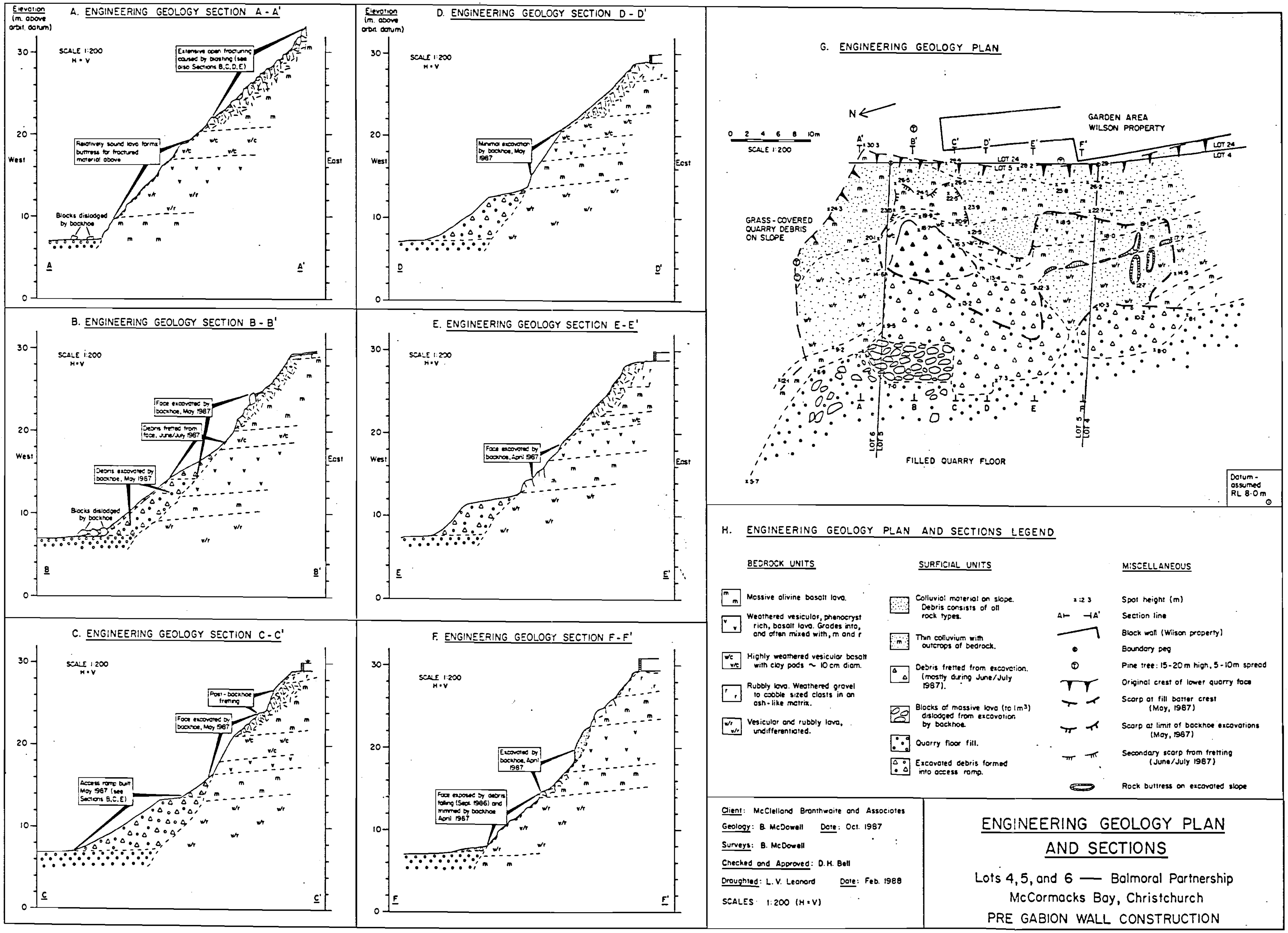

Engineering Geology Plan and Sections.
Lot 24 (Wilson Property), Glenstrae Ave,
McCormacks Bay, Christchurch.
Pre Remedial Work.
Figure 2.8
North
0 2 4 6 8 10m
Scale 1:200
Stone wall removed
Stone wall
Lot 22
Lot 24
Glenstrae Ave
Legend
Bedrock units
m massive lava. (note: see section 2.4.2 for descriptions of all units)
V/R vesicular + rubbly lava (undifferentiated).
R rubbly lava.
red ash/tuff.
Surficial Units
gravelly colluvium with outcrops of bedrock.
debri apron at base of slope
quarry floor fill
Miscellaneous
spot heights (m)
section line.
legal boundry + pegs.
Scarp of former quarry batter slope.
Survey : April, 1987
Geology : May, 1987 All B.J. McDowell
Drawn : March, 1989
Scale 1:200
H=V
Stone wall removed
Recent backing fill
detached block moved ~3m downslope
(m)
A
A'
Scale 1:200
H=V
Stone wall
backing fill
block source?
detached block
Fractures + cavities from blasting
B
B'
Composite log of quarry floor fill, Lot 24
(m)
Surface
Red brown, mod wthd, firm to compact, moist, sandy f-crs gravel with some clay + silt, some/rare boulders.
Fines mod plastic, give red colour.
Gravel f-crs, boulders to 60cm, mod wthd rubbly lava and phenocryst rich vesicular lava, clast angular.
Grey brown, slightly wthd, compact, moist, sandy f-crs gravel, some boulders, rare clay + silt.
Fines mostly coarse sand, sl to non plastic.
Gravel + boulders to 40cm, sl-mod wthd rubbly, vesicular + massive lava, clast angular.
Lt-med grey, sl wthd, massive olivine basalt
Phenocrysts of feldspar, olivine, rare amphibole.
Surface sharply stepped 'ridges' and 'gullys' formed by blasting (described by builder as seams of rock).

Figure 2.9 : Reduction of Pre Gabion Wall Plan and Sections



Ash and lahar deposits will only reach significant thicknesses when preserved in major gully systems. They are restricted to the lower slope on lots 3 and 4 (Balmoral Partnership, see Figure 2.10), and to the boundary of lot 22 and 24 beneath the stone retaining wall (Figure 2.8).

## 2.4 SITE INVESTIGATIONS.

### 2.4.1 Lot 22 (Glenstrae Ave).

The first investigations at McCormacks Bay were undertaken by D.H. Bell and P.J. Glassey (University of Canterbury) on Lot 22 (see figure 2.1) in 1985. Their investigation methods included: (a) mapping, and logging of exposed faces, (b) single channel seismic refraction profiles ,and (c) stadia surveys with detailed leveling. Plans, sections and descriptions were produced on a summary sheet (reproduced as Figure 2.11) which accompanied a short report to the consulting engineer.

The objective of this study was to ascertain the quality and extent of fill that had recently been placed behind a loose stone retaining wall, and also ascertain the stability of the wall for certification. It was concluded that the fill was adequate, but further work was needed to determine the stability of the wall foundations. The major recommendation was to remove the stone wall and build a retaining structure designed along sound engineering principles (Bell, 1985, unpublished report to Halliday O'Loughlin and Taylor).

### 2.4.2 Wilson Property (Lot 24, Glenstrae Ave).

Site investigations and stabilisation/remedial works were carried out on this property in April/May 1987 to allow the construction of Mr Wilson's house to proceed. This work occurred after removal of the stone wall on Lot 22 (above Lot 24, Figure 2.1), but before replacement with another structure, and before final clearance of loose material from the slopes of the Balmoral Partnership property below lot 24 (Figure 2.1). The timing of remedial works on the three separate lots that make up the quarry area was crucial in determining the type and extent of work that had to be undertaken (see also section 2.6.3 for discussion of idealised site investigation and remedial programme for entire quarry area).

Site investigations were aimed at assessing stability of the bench floor fill and the old quarry batter slope behind the house site. After an initial inspection a decision was made to produce a site plan from a

Figure 2.10 : Reduction of Balmoral Partnership Summary Sheet

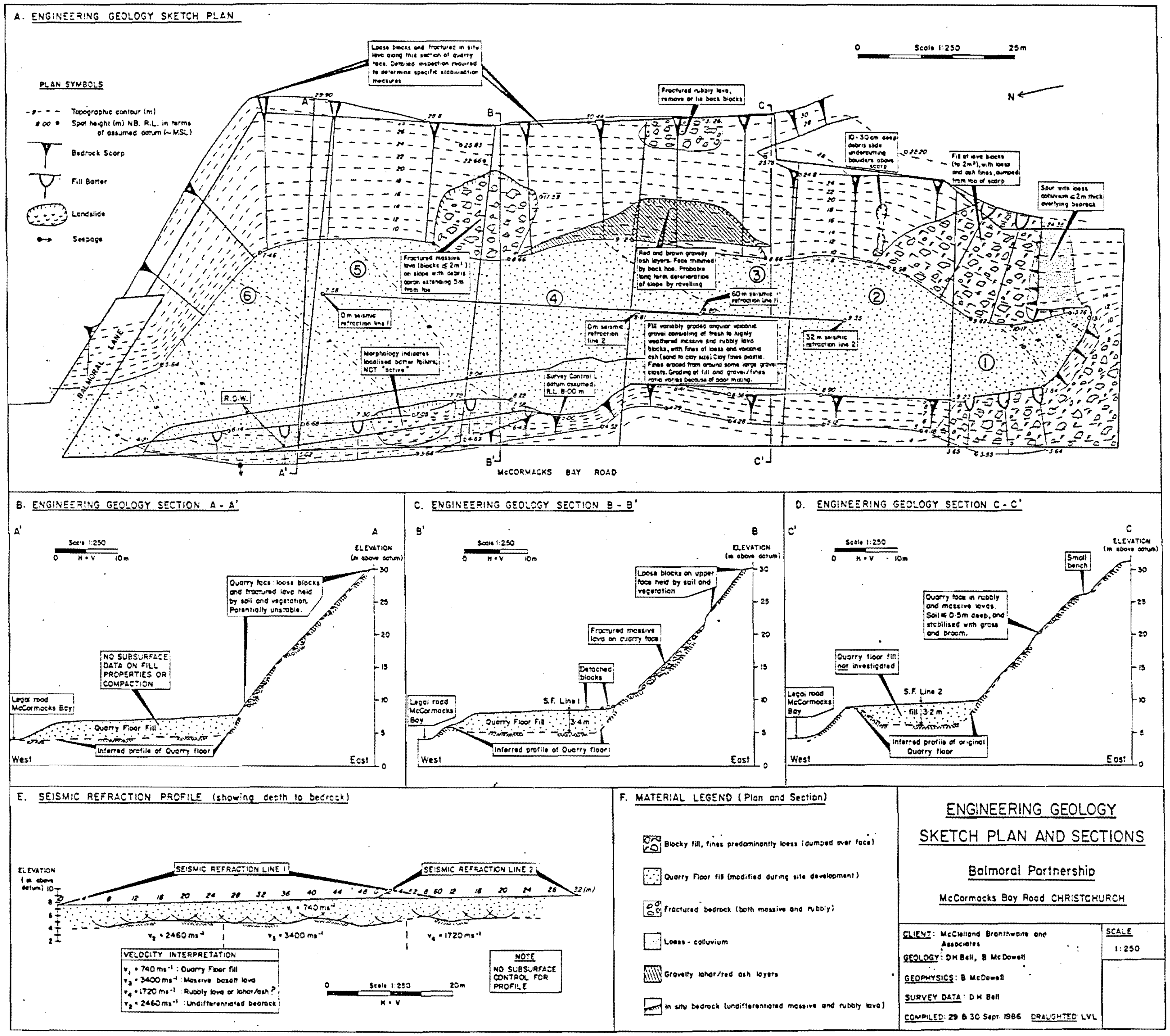
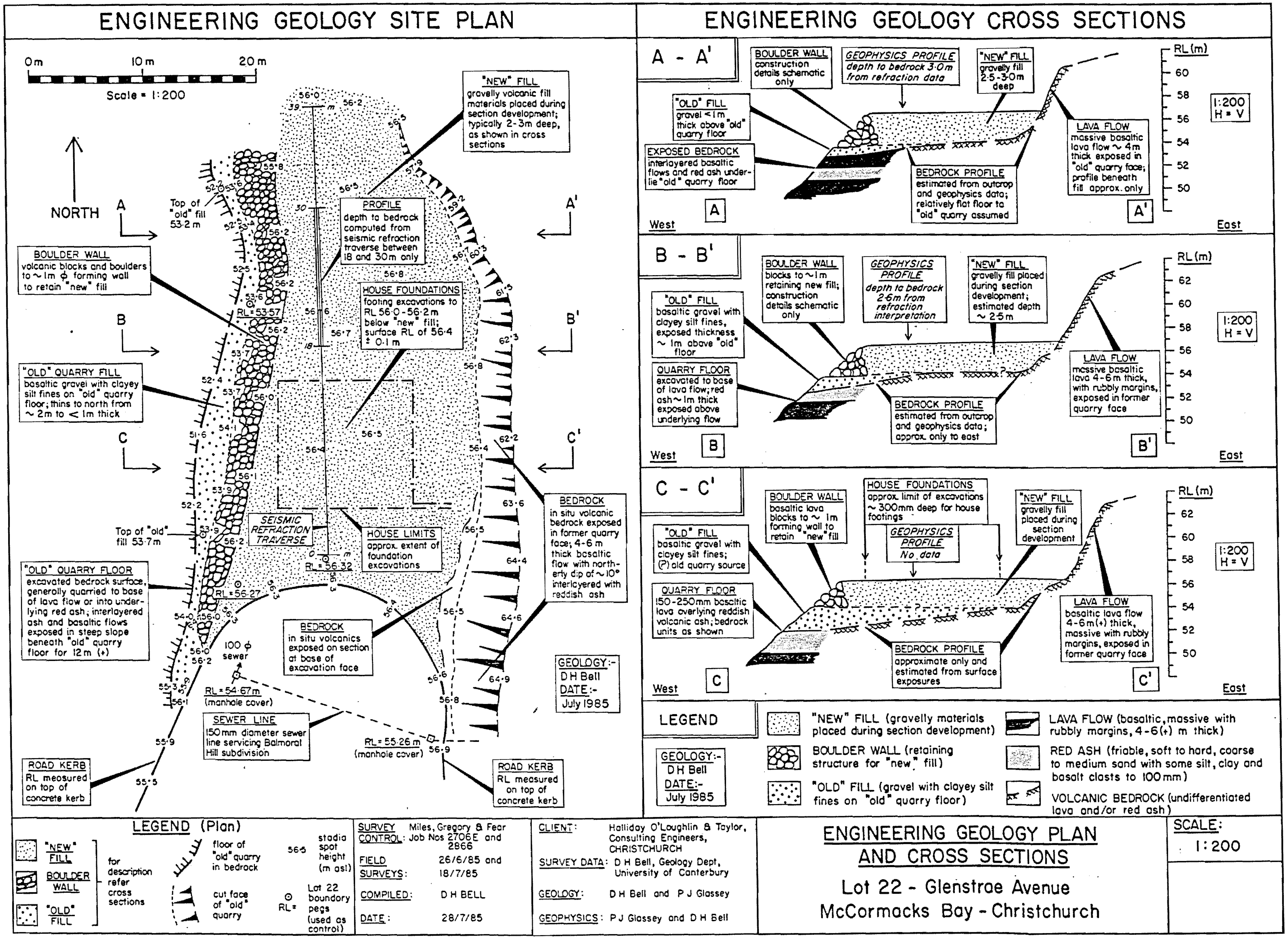


Figure 2.11 : Reduction of Lot 22 Summary Sheet (D.H. Bell pers comm 1988)



limited stadia survey. A drafted plan and sections were not required by the consulting engineers, so a very simple pencil drafted plan (Figure 2.12) was drawn up to aid in the written recommendations.

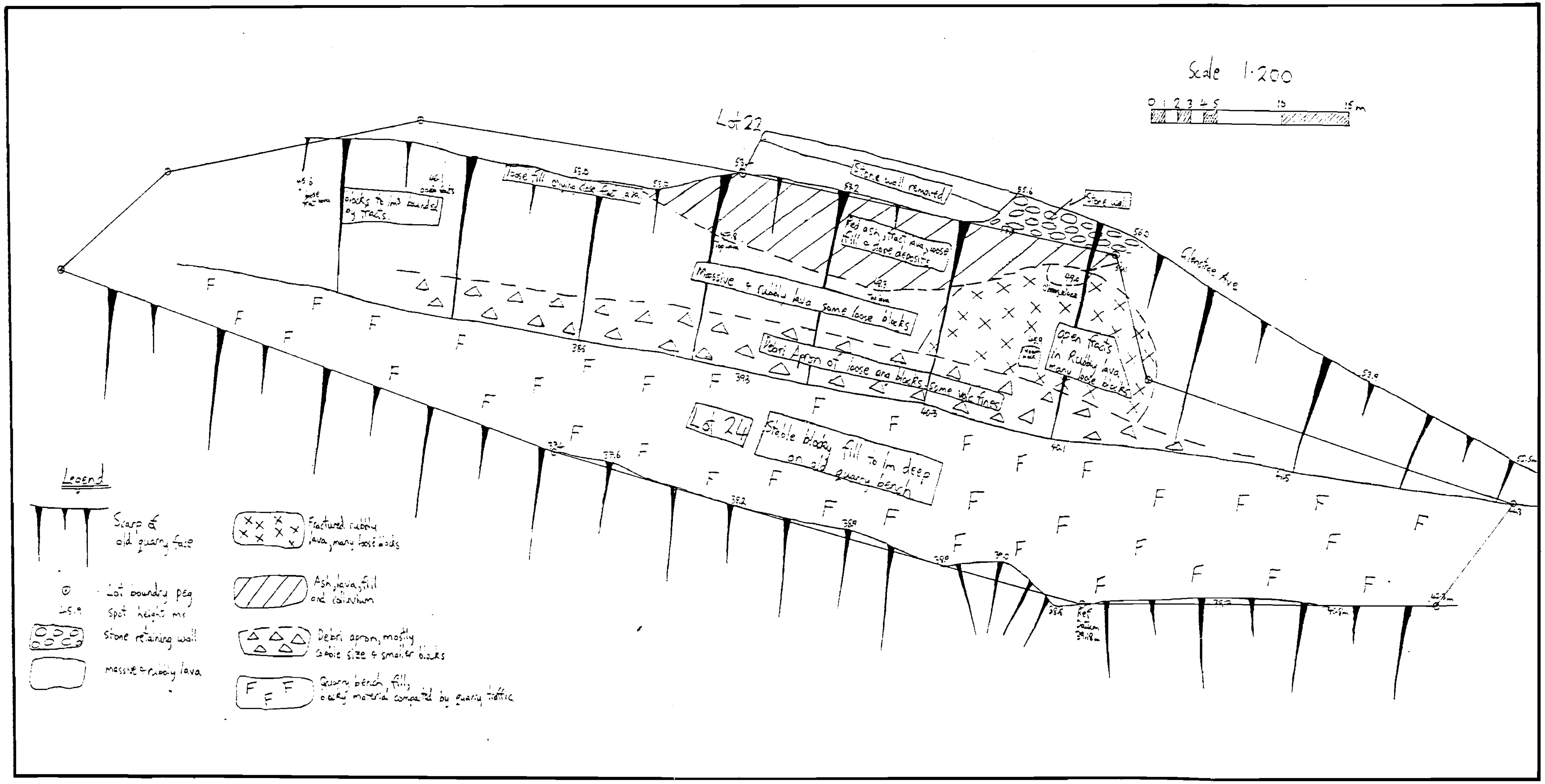
The first inspection and walkover was sufficient to identify the major problems without resorting to more detailed mapping, or other investigation techniques (e.g. geophysics, trenching). Areas of concern can be divided into house foundations, upper slope and lower slope. The lower slope is the steep batter slope of Balmoral Partnership and was looked at from the point of view of a failure at the crest undermining the Wilson house foundations. It was recognised that slope failures in colluvium and fractured bedrock could cut back upto 2m from the present slope crest. Inspection of the foundation area revealed outcrops of massive and vesicular lava, with blocky fill not more than 1m deep, depending on the profile of the original quarry floor. Areas of loose fractured material and colluvial debris were identified on the upper slope, and a site meeting was held with engineer, builder and owner to discuss remedial measures across the upper slope.

Minimum disturbance was the key factor in remedial works. Isolated removal of large blocks and a tree and shrub planting programme by Mr Wilson, along with a reinforced concrete block retaining wall and catch bench at the base of the slope, completed stabilisation work above the house site. A heavily fractured area of the slope at the southern end of the house site required clearing by backhoe and jackhammer, with buttressing of some large "semi-detached" blocks (see section 2.5.3 for details). Detailed mapping of lithologies and logging of exposed faces was carried out during these remedial works, with the main aim of determining quarrying practices which led to the present conditions (inferences about former quarrying practices are discussed in section 2.6.2.) Resurveying the upper slope, once remedial works were complete, was the last stage in documentation of this site (see section 2.5.3 for plans and sections showing remedial works).

### 2.4.3 Balmoral Partnership.

Mr D.H. Bell (thesis supervisor), with the author assisting, has been involved in investigations and checking progress of remedial measures at this site from September 1986 to August 1988. The work included: (1) preliminary stadia survey/engineering geological mapping and evaluation of instability, (2) on site discussions with both the consulting engineers and contractors regarding possible remedial measures, and (3) detailed

Figure 2.12 : Reduction of Original Working Plan for Lot 24
Site Investigations



pre-consturction logging and post-construction documentation for major remedial works (i.e. gabion basket retaining wall).

The original work (September 1986) was carried out to assertion the condition of the steep batter slope and the bench floor fill. Minor rock falls during August 1986 had undermined an area of fractured bedrock and loose colluvial material (Figure 2.13), leading the North Canterbury Catchment Board to question the long term stability of the batter slopes. Single channel seismic refraction was used to determine the approximate depth of fill, and a base plan (1:250 scale) was drawn up from a limited stadia survey. Engineering geological mapping involved a brief walkover, using the 1:250 base plan, and a final check before compilation of plan and sections. Figure 2.10 is a reduced version of the original summary sheet, which included an engineering geological plan, sections, the refraction profile interpretation, and material descriptions. Areas of fill material and potentially unstable slopes were annotated, together with general recommendations on remedial measures. Two main areas on the batter slope across lots 4,5,6 (Figure 2.10) were identified as needing some remedial work (actual extent of work not predictable) to remove or stabilise loose, blast shattered material, especially large (> $0.5m^3$) blocks of massive lava.

General rock types (e.g. fill, colluvium, bedrock) and areas of instability were identified on the plan, but it was not considered necessary to map details of such things as distribution of bedrock types. This sort of detail would have involved significantly more field work, including collecting more survey data, which could not be justified at that level of investigation. An example of the approach adopted is at the North end of the batter slope, where areas of blast fractured and loose massive lava were not differentiated (Figure 2.10), and instead a detailed walkover with engineer and contractor (to discuss the extent and details of remedial works) was recommended.

The favoured method of reducing the risk of rock fall from the batter slope was to remove loose blocks and colluvial debris from obvious problem areas. A backhoe, with a 5m long length of "I" section steel beam attached to the arm, pulled down loose material, but could not reach more than 1/3 of the height up the slope. Options for clearing the upper slopes included sluicing, using a crane with wrecking ball, and building a ramp from dislodged debris to allow backhoe access. A crane with wrecking ball was tried with minimal success, and then a backhoe moving up a self-made ramp of rubble and rock fill was used (see section 2.5 for more detailed discussion). The backhoe unearthed a deep pocket of loose, heavily blast

Figure 2.13 : Fresh Natural Failure in Loose Colluvium and Blast Damaged Bedrock (photo taken October, 1986)


Figure 2.14 : Limit of Fretting after Backhoe Excavations and Before Building of Gabion Wall. Note colluvial cone (from fretting of scarp) on backhoe ramp, and site of October,1986 failure (arrowed)

fractured massive lava, that had not been evident from the surface exposure .

The next stage of site investigation proceeded after a decision to build some sort of retaining structure across the scarp left by the backhoe excavations (Figure 2.14). A detailed site inspection by engineer, engineering geologist and contractor led to the adoption of a gabion basket wall, and a plan to produce detailed maps and sections across the area both before and after construction of the wall . Investigations involved a detailed topographic survey using an electronic theodolite and distomat, followed by recording of lithologic and engineering geological features and contacts onto a base map. A summary sheet was produced that included several close spaced sections to illustrate the changing conditions across the area of concern (Figure 2.9). Features highlighted on the summary sheet included the limits of fill material, backhoe excavations, and fretting of excavations. Bedrock was divided into readily recognisable local units related to the degree of weathering and lava flow morphology (see section 2.3.3). Note that the position of Lot boundaries provide a reference between figure 2.9 and 2.10.

A site visit was made towards completion of benching the vesicular lava to found the first gabion baskets, to check for any problems in the foundations. Possible problems could have included extensive open fracturing, water seepage and severe weathering, but the vesicular lava proved to be particularly sound material (see sections 2.3 and 2.5.4 for further discussion of vesicular lava and foundation conditions).

The final fieldwork at this site involved re-surveying the lot 4,5,6 area after completion of the gabion wall and final tidying across the excavated area. A summary sheet similar to the preconstruction sheet was drawn up to allow direct comparisons between before and after construction profiles across the excavated area (Figure 2.15).

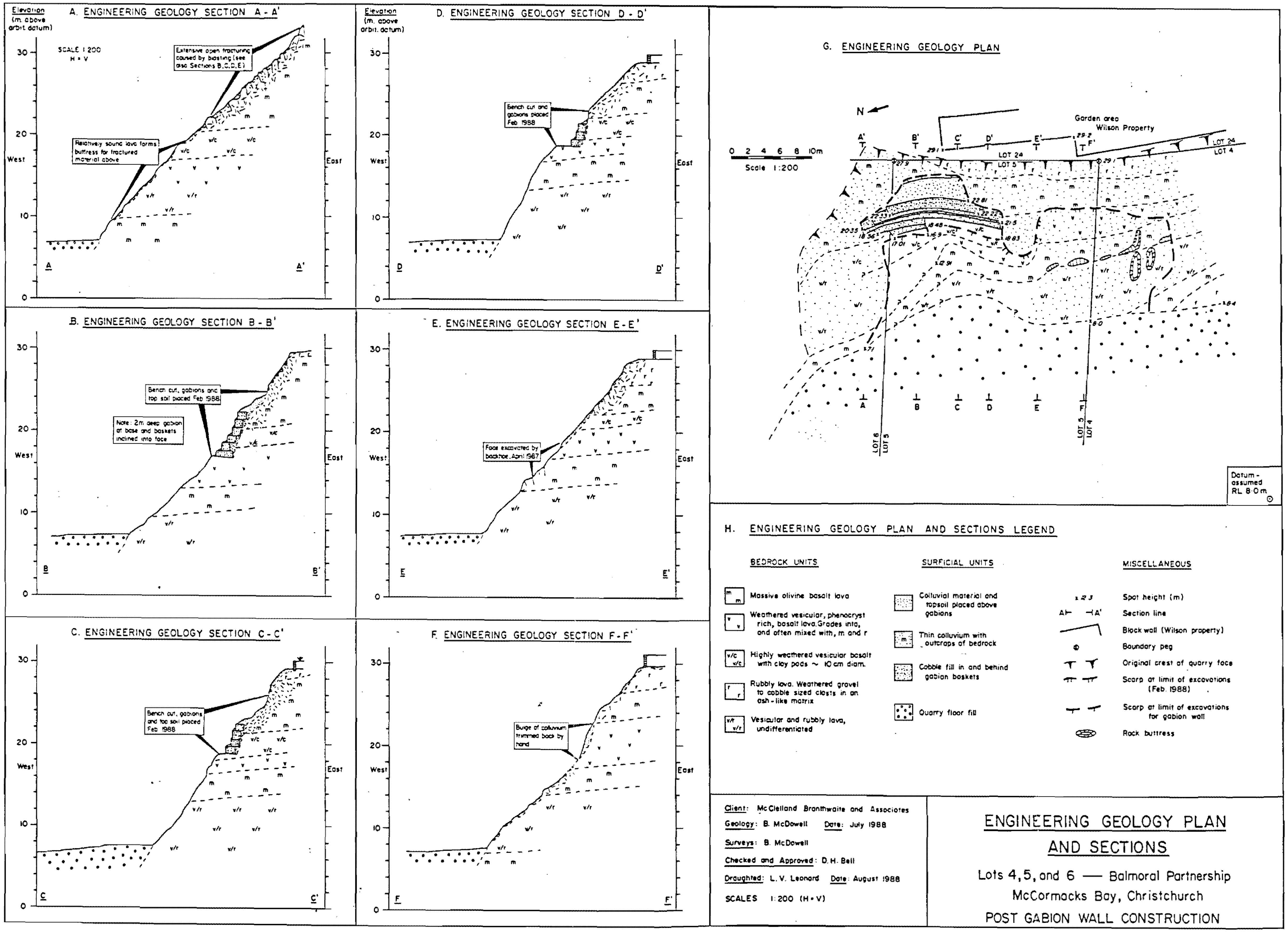
## 2.5 REMEDIAL MEASURES.

### 2.5.1 Design Approach.

As outlined in section 2.1 these investigations and remedial measures were carried out after subdivision of the disused quarry into three major properties.

The small scale nature of the stabilisation work on the McCormacks Bay Quarry, along with the need for a low cost solution that could be

Figure 2.15 : Reduction of Post Gabion Wall Plan and Sections



financially absorbed by the property owners, meant that a design-as-you-go approach (Fookes & Sweeney, 1976) was adopted. Remedial measures were initiated after a minimum of investigations and with no real knowledge of the sub-surface conditions. Close supervision of works by the engineering geologist allowed for modification to the originally proposed remedial measures as more of the face was exposed.

Decisions on the relative stability of colluvium and bedrock on the quarry batter slopes was a totally empirical process. Conventional rock mechanics approaches for jointed rock slopes (e.g. Hoek & Bray, 1981) were not applicable because of the interlayering of bedrock units with different material properties and fracture characteristics, as well as the added complication of blast fracturing. Defining the geotechnical properties for 'massive', 'vesicular' and 'rubbly' materials, and the different styles of fracturing was well beyond the scope of such an investigation. Defining persistent joint and fracture orientations in blast-affected rock was also an unrealistic, if not impossible, task.

Deeply fractured insitu bedrock aside, the faces were covered in a loose gravelly colluvium, with large blocks sitting on the surface, or partly embedded in the colluvium. Large blocks upto $2m^3$ were the most obvious danger to any residential developments below. The nearest analogy to this situation in the literature are cases of risk of boulder falls onto housing estates in Hong Kong.

Two papers (Threadgold & McNicholl ; Grigg & Wong, 1987) cover the range of investigation and remedial options. Large fields of boulders (upto $120m^3$ in size individually) require an economic balance between protection works and a minimum of individual boulder stabilisation. Options for protection include rock trap ditches, mass barriers (e.g. gabion type structures -Threadgold & McNicholl) and wire catch fences where space is at a minimum, while well established vegetation is useful in preventing major movement of small boulders and gravel. Stabilisation of boulders may be effected by: (a) breakdown and complete removal, (b) reduction in size of boulder by trimming, (c) buttress and prop, (d) dowelling of jointed boulders. When discussing mathematical modelling of boulder stability and trajectory/velocity, Threadgold & McNicholl state: "it is clear that many factors influence boulder behaviour and the variables of any site would require an extremely large modelling and testing programme. Even then a large number of unknowns would remain." Likewise, Grigg & Wong conclude that "assessment of boulder stability was not used for remedial measure design due to inherent problems of determining three dimensional boulder shape and assessing underlying founding conditions." A qualitative approach to stability requires a conservative approach to remedial measures, but no more so than an incomplete quantitative approach with assumed values for many variables.

2.5.2 Lot 22.

Lot 22 (the highest bench, see figure 2.1) was investigated in 1985, and it was recommended that a loose stone wall, retaining recently placed blocky fill (see figure 2.11), be removed and replaced with a correctly designed and founded structure. The wall was removed in late 1986, although an alternative structure is yet to be built. Unretained fill on the undeveloped lot has shown no sign of instability in the last 2 years. Construction of a new retaining structure, to ensure long term stability of the filled bench, will be complicated by the need to protect the Wilson house (on the bench below) from damage by falling debris.

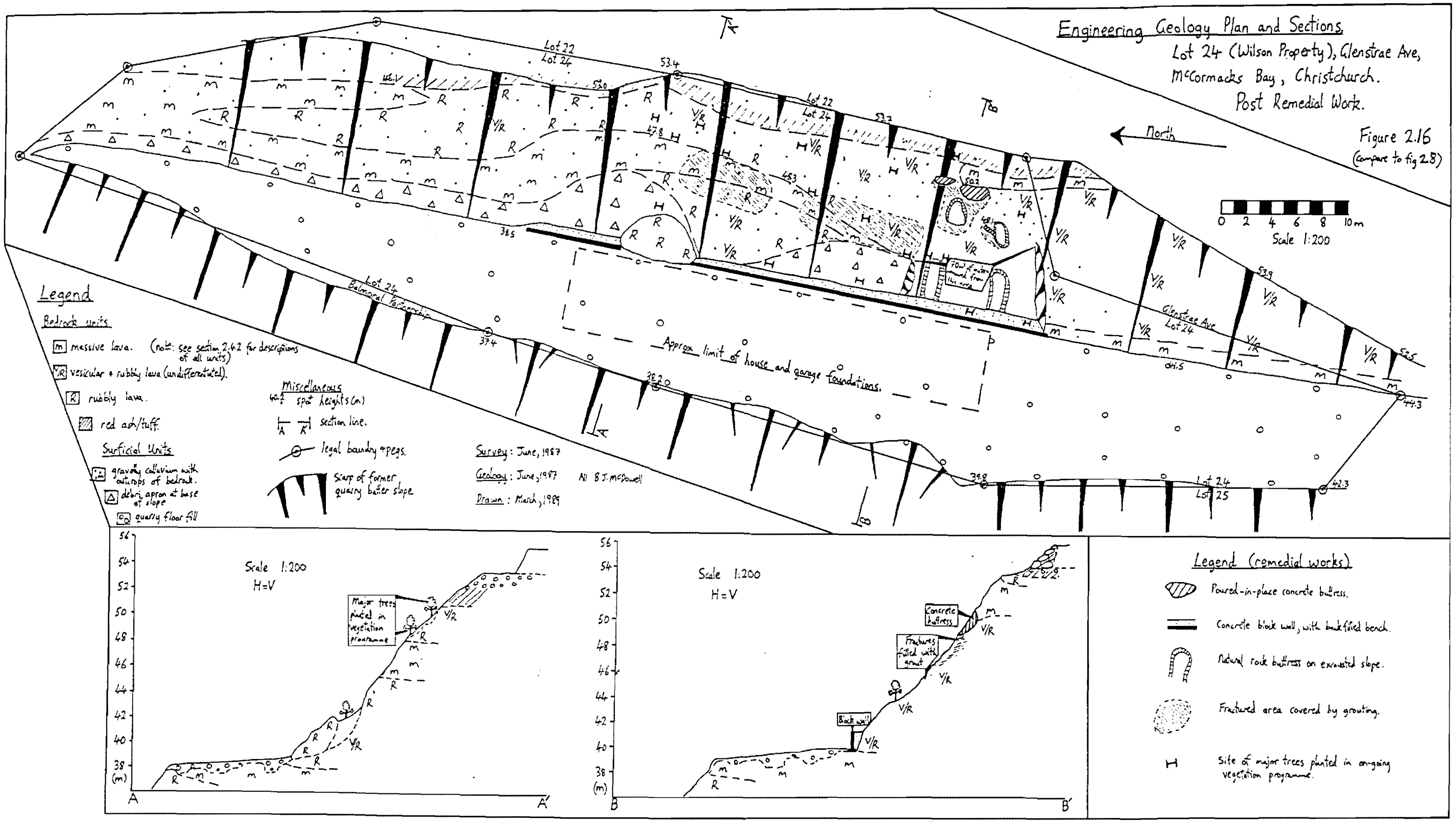
2.5.3 Wilson Property (Lot 24).

Investigations on lot 24 were started in April 1987, with remedial measures completed in May 1987. Remedial works were aimed at decreasing the risk of block and debris falls and slides from the upper batter slope. Completely clearing the slope of fracture-bounded blocks and loose gravelly debris/colluvium was not possible because of the risk of undermining the Lot 22 boundary and Glenstrae Ave itself at the southern end of the face. The overriding principle was minimum disturbance of the slope, with the acceptance of a continuing risk of small slides and falls of debris (with maximum size clasts of approximately 200mm).

Initial remedial work was discussed at a site meeting between the engineering geologist, owner (Mr Wilson), architect, builder/contractor and certifying engineer. Removal of isolated loose boulders by hand and clearing of loose material off the heavily fractured southern area of the batter slope were adopted as a starting point for remedial measures. As work continued, further remedial measures were used, and these included : (a) grouting of open fractures and cavities, (b) trimming of large blocks, (c)buttressing of blocks by poured in place concrete, (d) tree and shrub planting, and (e) building of a concrete block wall and catch bench at base of batter slope.

The following list outlines the sequence of events in the remedial programme (see Figure 2.16 for positions of remedial works).

(1) Bar down loose blocks upto $1m^3$ from face. Loose blocks were



Engineering Geology Plan and Sections,
Lot 24 (Wilson Property), Glenstrae Ave,
McCormacks Bay, Christchurch.
Post Remedial Work.
Figure 2.16
(compare to fig 2.8)
North
Scale 1:200
0 2 4 6 8 10m
Lot 22
Lot 24
Lot 25
Glenstrae Ave
Approx limit of house and garage foundations.
Legend
Bedrock units
massive lava. (note: see section 2.4.2 for descriptions of all units)
vesicular + rubbly lava (undifferentiated).
rubbly lava.
red ash/tuff.
Surficial Units
gravelly colluvium with outcrops of bedrock.
debris apron at base of slope
quarry floor fill
Miscellaneous
spot heights (m)
section line.
legal boundary pegs.
Scarp of former quarry batter slope
Survey: June, 1987
Geology: June, 1987
Drawn: March, 1989
All B.J. McDowell
Scale 1:200
H=V
Major trees planted in vegetation programme
Concrete buttress
Fractures filled with grout
Block wall
A
A'
B
B'
Legend (remedial works)
Poured-in-place concrete buttress.
Concrete block wall, with backfilled bench.
Natural rock buttress on excavated slope.
Fractured area covered by grouting.
Site of major trees planted in on-going vegetation programme.

restrained with straps and ropes, while being prised free from the slope.

(2) Clear about $70m^3$ of material from the southern fractured area by backhoe, and truck excess spoil to fill, or use as for property access).

(3) Detached blocks and ribs exposed on fractured area are trimmed to more stable configuration with jackhammer.

(4) Non-pressurised grout (approximately $2m^3$) placed in open fractures and cavities along central part of batter slope. Non-pressurised grout was intended to decrease the amount of surface water flow into cavities, and cement together fracture bounded blocks without reducing stability of the slope by jacking blocks out of the slope (possible with pressurised grouting).

(5) Approximately $1m^3$ of grout placed on north edge of southern fractured area.

(6) Three steel-reinforced concrete buttresses poured under potentially unstable blocks (Figure 2.17) , and $1m^3$ of grout placed in fractures about buttress foundations. Concrete buttresses were tied to the slope by driving reinforcing into the 'rubbly lava' and grouting up fractures at the base of buttresses. The rockface was trimmed, then cleaned with compressed air prior to pouring of buttresses.

(7) Reinforced concrete block wall (2m high) founded on bedrock along toe of batter slope, and backfilled to create catch-bench for small falls of gravelly debris. The reinforced block wall incorporated a cut-off drain to intercept surface flow from the batter slope before it can infiltrate the bench floor fill.

(8) Planting of trees and shrubs across face to minimise risk of failures in loose gravelly colluvium. A well maintained cover of vegetation on the slope serves to bind the loose colluvium and provide a barrier for any minor falls of gravel sized, and smaller material (points 6,7,8 are shown in Figure 2.18) .

No work was carried out on the lot 24/Balmoral Partnership boundary, because house foundations were kept at least 3m from the edge of the lower batter slope above lots 3 and 4 (Balmoral Partnership), where exposed bedrock has only minor blast fracturing.

### 2.5.4 Balmoral Partnership (Lots 1-6).

Remedial works on this property were aimed at removing loose colluvial material and blast-fractured bedrock from the batter slopes of lots 4,5,6. The inability to obtain detailed subsurface information led to a lack of knowledge about the style and extent of slope failures that could occur

Figure 2.17 : Boxing in Place for Buttressing Massive Lava Blocks on Lot 24 Batter Slope

Figure 2.18 : Concrete Block Wall, Concrete Buttresses and Tree Planting Completed on Lot 24, South End

from slope saturation or earthquake loading. Such knowledge would be a necessary consideration for any design of restraining or protection works. The large number of fractured blocks also makes stabilisation by restraining methods (e.g. buttresses, dowelling) unrealistic. Protection of the lower bench against individual block falls by catch fences or ditches would be possible, but not effective for a slope failure involving a large volume (e.g. $100m^3$) of blocks and gravelly material. Removal of potentially unstable material was seen as the best option, given the expense of obtaining further information by more extensive investigations.

Remedial works, spanning an 18 month period from September 1986 to February 1988, are outlined in order below.

(1) August 1986: uncemented ash and lahar layers exposed in the lower batter slope on lots 3 and 4 were trimmed by a backhoe (see figure 2.10).

(2) December 1986: a backhoe, with an I section steel girder extension, was used to "prod" down loose blocks and debris from around the original slope failure (see figure 2.13) and the potentially unstable area on lots 5 and 6 (see figure 2.10, section A-A'). The method was successful, but the backhoe could only reach about 1/3 of the batter slope and fractured loose material clearly continued higher onto the slope.

(3) May 1987: an attempt was made to clear the higher slopes using a crane and wreaking ball. This experiment was only partly successful, as the ball lacked any "plucking" effect when dragged up the slope and could not be controlled well enough to allow trimming of the face. High pressure sluicing was considered as a fast and efficient method of cleaning down the batter slope, but there were no contractors in Christchurch with equipment or expertise in this field, so the idea was abandoned.

(4) May 1987: excavation of loose material continued on lot 5, with a backhoe accessing the higher slope by a ramp built from debris already pulled from the face (see Figure 2.9). A pocket of deeply fractured massive lava was uncovered and upto $10m^3$ of material was being brought down with each stroke of the backhoe. Excavations were halted when they threatened to undermine the garden wall of the Wilson property above (see figure 2.14).

Excavation of loose and fractured material from a slope by a backhoe working from below (as opposed to working down from the top) tends to give a vertical to overhanging working face, which leads to less control on the limit of trimming. The practicalities of this site dictated that backhoe excavations were carried out from the base of the slope. A very close liaison between supervising engineering geologist and the backhoe operator may be required to avoid problems similar to those encountered on lot 5, where the scarp was excavated too deeply into the fractured 'massive lava' before close inspection of the unexpected exposure could be made.

(5) October 1987: a site meeting was held between engineering geologist, consulting engineer and contractor to discuss options for a retaining structure to stabilise the over-steepened scarp on lot 5. There was an obligation to stabilise the slope without any more major excavations, because the Wilson property was already developed and needed protection from any retrogressive failure of the batter slope below.

Options for a retaining structure on lot 5 included: poured in place concrete, prefabricated concrete, crib wall, and gabion basket. Major problems for the designing engineer were prediction of foundation quality and estimating loads that a wall may have to withstand. A third problem was difficulty of access. These three problems all led towards a gabion basket wall as the preferred structure. Leventhal and Mostyn (1987) site ease of construction and founding on akward sites as prime advantages of gabion basket walls. The inherent flexibility and good drainage are also useful attributes. Flexible design allows for settlements of the face without failure of the structure.

Settlements are acceptable on the batter slope of lot 5 (A. Cochran pers comm 1987) , but estimating the extent of such forces and designing a suitable inflexible structure (e.g. poured in place reinforced concrete) to be founded on the face would be difficult, time consuming, and expensive. The maintainence of good drainage on the face was also considered essential to stability of the openly fractured 'massive lava', which has ample source for water infiltration from the Wilson property above. Gabion baskets filled with cobbles and back filled with gravel are at least as permeable as the fractured massive lava they support. Foundations for the gabion structure only require excavation of a back tilted bench, with minimum anchoring back into the volcanic bedrock, which may not afford reliable anchoring on a more extensive scale (e.g. as required for a rigid concrete retaining wall).

(6) February 1988: gabion wall erected. The wall was founded on benches cut in unfractured 'vesicular lava' (see figure 2.5). Some of the more precarious blocks on the scarp were pulled down using a backhoe and wire rope to make the site safe for excavation of the benches by jackhammer and light blasting. The baskets were placed, filled with cobbles, and the wall backfilled with gravel by a backhoe reaching from the fill ramp. Topsoil was placed, on the bench created above the gabion wall, to allow for the establishment of shrubs and groundcover vegetation. Figures 2.9 and 2.15 show pre- and post-gabion wall construction plans and sections over lots

4,5,6. Figure 2.19 gives a view of the completed gabion wall before removal of all the excavated debris (compare with Figure 2.14).

(7) July 1988: final trimming back of bulges of colluvial material along the edge of excavations on lot 5 and removal of excess spoil from the site.

## 2.6 DISCUSSION.

### 2.6.1 Mass Behaviour of Engineering Geological Units.

Field observation and description indicate variations in material properties of the various units, such as density, permeability, compressive strength and durability. During the early stages of work on the site a laboratory testing programme was considered, but later discounted when it was realised that mass properties (i.e. style and degree of fracturing) were the controlling factors in the relative stability of bedrock units. The very coarse nature of much of the fill material limited the useful geotechnical information that could be obtained from these units.

Ash/ lahar deposits are uncemented, but compact and essentially unfractured , providing adequate bearing capacity on benches and stability in steep batter slopes. Exposed (i.e.unvegetated) faces of ash and lahar do suffer minor fretting and rill erosion (with associated small rock and debris falls) from surface water run-off.

'Vesicular lava' and blocks from 'rubbly lava' have similar material properties, but compared to 'massive lava' they are softer (to the point of being crumbly) and weaker in compression. The proportion of vesicles and weathered phenocrysts, as well as density and porosity, appear to be the controlling factors in material strength. This relative material weakness actually aids in stability of the rockmass in steep slopes, because a lack of natural fractures, such as cooling joints, and the soft/crumbly nature of 'rubbly' and 'vesicular lava' make it resistant to fracture opening and shattering from blasting in a quarrying situation. However, the few persistent fractures that develop from blasting may isolate large unstable blocks, if orientated unfavourably. The apparently favoured method of blasting (i.e. the bulled hole method, section 2.6.2.) at McCormacks Bay has lead to fracturing and unstable blocks of 'vesicular lava' on Lot 24 immediately south of the house site. This is the only area where there is evidence of heavy blasting in 'vesicular lava' (an area of similarly fractured 'rubbly lava' occurs at the North end of the Lot 24/Balmoral boundary) and it required most of the remedial efforts on Lot

Figure 2.19 : Completed Gabion Wall. Wilson house and garden wall are on bench above

Figure 2.20 : Blast Fractured Massive Lava. Large blocks are bounded by opened cooling joints

24 (see section 2.5.3).

Thick 'massive lava' units are characterised by cooling joints, which are steeply dipping fractures with a polygonal pattern in plan view. When exposed in a face or slope the dominant joints fall into 2 sets striking approximately parallel and perpendicular to the face. Natural faces, such as sea cliffs, are prone to block falls controlled by cooling joints when individual lava flows are undermined by fretting of uncemented ash/lahar layers. The sea cliff at Clifton beach (Sumner) is an example of natural rock fall modifying development in the area. Around the turn of the century the tramway and road to Sumner was blocked several times by rock falls from the cliff, eventually leading the tramway company to construct a causeway between Shag Rock and Sumner to by-pass the danger area (Ogilvie, 1978).

The dense, fine grained nature of 'massive lava' makes it resistant to fracturing and disintegration by natural weathering processes. However, the same properties promote fracture and shattering into angular pieces when subjected to heavy blasting. The major remedial works on lots 4-6 (Balmoral Partnership, figures 2.9 and 2.15) were carried out in an area of heavily fractured 'massive lava'. Figure 2.5 shows the brittle nature of the 'massive lava' compared to that of the underlying 'vesicular lava'. A close up view of fractured 'massive lava' at the top of lot 4-6 excavations (figure 2.19) highlights large blocks bounded by cooling joints, with angular gravel sized blocks formed by blast fracturing. The insitu blocks form an apparently stable interlocking mass, except for the occasional floating block not embedded in the slope.

Removal of buttressing support at the toe of a fractured area allows large blocks to relax downslope, increasing the risk of rockfall from on-going fretting of the slope (small falls, 10-20$m^3$) and major earthquake loading (possible large volume falls, upto 100$m^3$). The area of 'massive lava' supported by the gabion wall (figure 2.15) is similar to the fractured 'massive lava' immediately to the north (where no remedial works were carried out), except that the area to the north is buttressed by a steep slope of relatively unfractured massive, vesicular and rubbly lava (section A-A', figures 2.9 and 2.15). Toe support in the area of remedial works consisted of fractured bedrock and colluvium similar to the material that was involved in the natural slope failure of 1986 (figure 2.13). This material was not considered to have adequate long term support, as a relatively small debris failure could leave the rest of the slope in an undermined, unstable state.

A description of 'quarry floor fill' (given in section 2.3.2) indicates the material has a wide size range and variable grading because of a lack of mixing before or during placement. Wherever observed the fill is clast supported and compact, with some horizontal layering. Quarry spoil was probably dumped as it was produced, and compacted by vehicle traffic to create a stable material with adequate bearing capacity for suburban dwellings. Fill on Lot 24 (Wilson property) was estimated at upto 1.5m deep, and it was recommended that house foundations be keyed into bedrock to circumvent any problems from failure of fill at the outside edge of the bench. A composite log of the foundation excavations is produced in figure 2.8.

Blocky fill is quarry spoil similar to that used for quarry floor fill. The material dumped on the face above lots 1 and 2 (Balmoral Partnership) contains a significant amount of large boulders, which along with the gravel portion forms an apparently stable interlocking mass. There is no evidence of any slope movement or debris fall since the fill was dumped, however a caution against modification of the toe area on lot 2 was given in the final assessment of Balmoral Partnership (Bell, 1988).

Batter slope colluvium varies in thickness and composition across the site, but is generally a loose, open deposit of bedrock-derived gravel clasts. Colluvium is prone to fretting and small debris falls, especially on very steep faces (upto $80^0$) without adequate surface retention by vegetation. Apart from some trimming of colluvium on steep faces (e.g. see section F-F′, figure 2.9 and 2.15) a programme of shrub and groundcover planting (and maintainence) was advocated as an acceptable stabilisation measure (Bell, 1988).

### 2.6.2 Blasting Practices Deduced From Field Observations.

The existence of blast fractured and loosened material in considerable quantities on the quarry batter slopes was the major reason for engineering geological investigations and the subsequent remedial works. Blast fracturing is most severe in thick 'massive lava' deposits and adjacent 'vesicular' and 'rubbly lava'. 'Massive lava' is the strongest and most durable bedrock unit, and was sought out as the favoured rip-rap and fill material for sea wall and causeway construction.

Evidence for blast hole fracturing was initially uncovered during mapping, and after excavations on the batter slope of Lot 24 (Wilson property). Bedrock on the southern part of the Lot 24 batter slope is a thick sequence of dominantly 'vesicular lava', with discontinuous layers of massive and rubbly lava. There may be two or more stacked flows, but no clear flow boundaries were observed across this part of the face.

Vesicular lava has had severe blast damage compared to other parts of the quarry. Blocks from several $m^3$ to gravel size littered the slope, with steeply dipping fractures open upto 150mm common. Some fractures lead to cavities of $1m^3$ and possibly larger. Several open drill holes of approximately 10cm diameter (presumably drilled for loading with explosives) were also noted while mapping the slope.

Removal of loose material from the southern part of Lot 24 (see section 2.5.3) gave an incomplete section of the face to a maximum depth of 2m. Large blocks of massive and vesicular lava are interspersed with blocks and wedges of shattered-in-place lava (gravel size and smaller material) at the base of the slope. Fractures in the relatively sound blocks occur at all orientations, but the significant fractures (in terms of block stability) are opened cooling joints and blast fractures both perpendicular, and parallel to the slope. If the face in front of a block is oversteepened, fractures subparallel to the slope may daylight and increase the potential for block movement. Figure 2.21 is a section through the base of the Lot 24 slope showing in place fractured material and a fracture bounded block of 'massive lava'.

Conclusive evidence of the style of blasting was found about half way up the Lot 24 batter slope, where a large rib of 'vesicular lava' was left after clearing of surface material from the slope. The surface of the block was horizontal at the top, dipping down to follow the angle of the rest of the slope (i.e. approximately $30^0$). Vertical open fractures bounded the back of the block, while open fractures subparallel to slope daylighted at the base. A vertical drillhole entered the block on its horizontal surface. Reducing the size of the block by 'chiselling' with a jackhammer revealed a cavity of about $1m^3$ (about 1/3 filled with completely shattered material) directly beneath the drillhole. Figure 2.22 shows the block and cavity after trimming with the jackhammer. Fractures, including those perpendicular and parallel to the slope, radiate from the base of the cavity. Note that fractures subparallel to the slope may curve upwards towards the surface at distance from the blast center, there by creating a relatively stable block (see sketch Figure 2.23).

The preceding description of the Lot 24 batter slope indicates a style of blasting that involved large explosions in wide spaced holes to produce large blocks and highly fractured material. It is clear that large amounts of blast-fractured material could remain on the slope after a round of blasting. The only quarrying technique that fits these observations is the heading blast, or bulled method. Grimshaw and Poole (1983) state that the

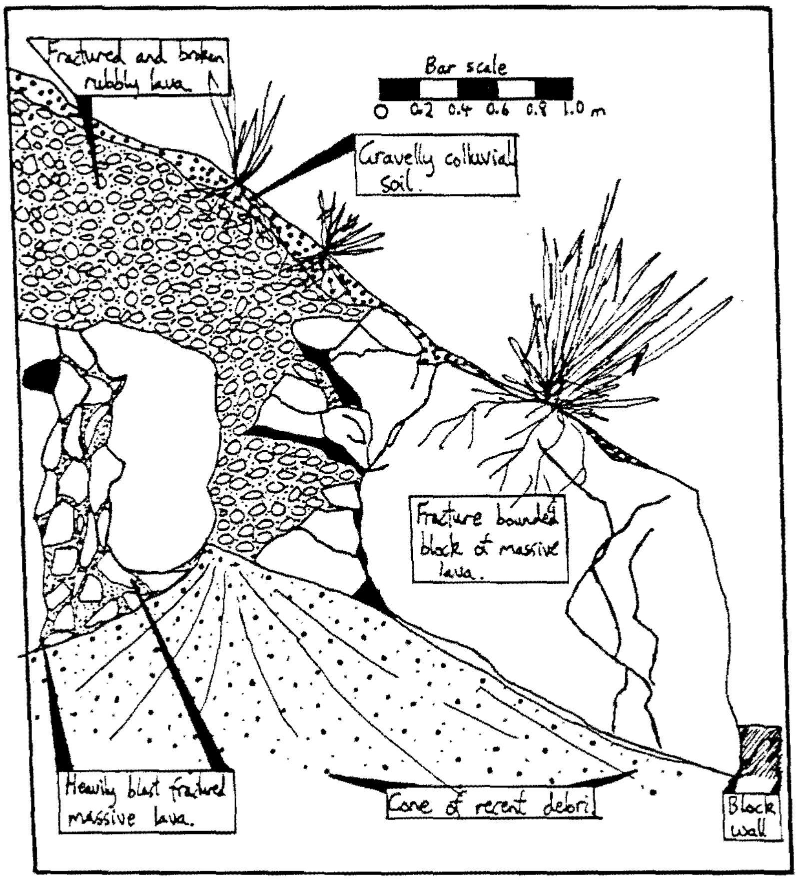



Figure 2.21 : Face Log at Base of Batter Slope, 5m South of Section B-B' (fig. 2.8). Massive block is buttressing fractured material

Figure 2.22 : Blast Crater in Vesicular Lava, Lot 24 Batter Slope. Most of detached block has been trimmed by jackhammer

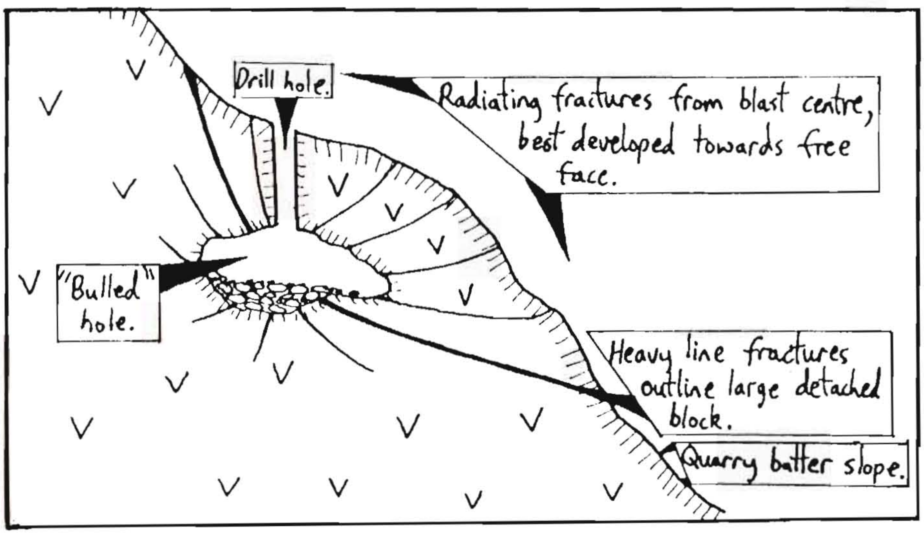



Figure 2.23 : Diagrammatic Section of Bulled Hole Blasting in Vesicular Lava Showing Large Detached Block

heading blast method was freely used in the United Kingdom in the 1940s and 1950s -"the method involved detonation of large quantities of high explosive in chambers excavated inside the rockmass in order to produce massive quantities of broken material, a large proportion of which had to be subsequently broken down by secondary blasting using 'pop' or plaster shooting." The "Handbook on Quarrying" (Department of Mines, South Australia, 1961) called the excavation of chambers 'bulling'. Relatively short, small diameter boreholes were 'bulled' by detonating increasingly larger charges of explosives at the bottom of the hole. When a large enough chamber is formed it is packed with explosives and fired simultaneously with other holes across the face. In comparing the 'bulled' method with the then recently introduced long hole, or full face method, the "Handbook" notes that : "the bulled hole method called for a great deal of experience and judgement on the part of the driller and powder monkey, together with a knowledge of local rock condition ... Good fragmentation was rarely obtained with the bulled hole method, but it can usually be obtained with the deep hole method." The deep hole (or full face) methods developed with the introduction of faster and more efficient drilling machinery. Prior to this the bulled hole method aimed at producing the maximum tonnage per foot of drill hole.

Three major reasons can be advanced for the probable retention of the bulled hole method at McCormacks Bay Quarry : (a) methods were never updated because of equipment costs, (b) large block sizes were required for rip rap material, and (c) difficult drilling conditions made full face drilling an unattractive (uneconomic) prospect. Reasons (b) and (c) seem the most likely, especially reason (c), as drilling blast holes in jointed 'massive lava' interlayered with cobbly 'vesicular' and 'rubbly lava' can be a difficult and time-consuming job (B. McGiffen pers comm *).

A major problem with the bulled hole method is getting the correct hole spacing and loading of explosives. Too little charge gives insufficient fracturing to break material off the face, while too much explosive results in fly rock and overshattering that locks much of the material onto the face. Excessive amounts of explosive fired instantaneously in a series of bulled holes appears to be the reason for large amounts of fractured rock encountered on parts of the quarry batter slopes.

(* Mr B. McGiffen is an experienced blasting contractor who was involved in constructing the gabion wall on Balmoral Partnership.)

2.6.3 Idealised Remedial Programme.

An idealised investigation and remedial programme is worth considering, because of compromises in the actual programme due to timing of events and cost considerations.

Remedial measures on the Wilson property (lot 24) could not reduce the risk of block falls or colluvial failure as far as might have been possible because care had to be taken not to disturb Glenstrae Ave or the filled bench of Lot 22 above the batter slope. The overriding factor was the need to keep cost, which was being born by the purchaser of the lot (Mr Wilson), to a minimum, as individual section owners generally have a limited capacity to pay for slope stabilisation measures above the cost of purchasing the lot.

Building the gabion wall on Lot 5 (Balmoral Partnership) was an expensive (approximately $40 000, A. Cochran pers comm 1988) and not entirely satisfactory way of stabilising the face of heavily fractured 'massive lava'. The simpler and less expensive method of stabilisation would have been to continue excavating the slope back to unfractured material. However, continued excavations were not possible as they would have encroached on the northern end of the lot 24 bench, where Mr Wilson had already developed a raised garden area. In this case timing considerations (i.e. the higher bench - Lot 24 - had been sold and developed before remedial works were finalised on the slope below) forced a more expensive than necessary option onto the Balmoral Partnership owners.

The most efficient and cost effective way of tackling investigations and remedial works on a disused quarry targeted for residential development would be to treat the whole quarry as a single site before subdivision and sale of lots. Site investigations, similar to those carried out for the separate lots on McCormacks Bay Quarry, are sufficient to delineate areas of potentially unstable material on the batter slopes, and remedial measures could then concentrate on removal of all fractured and loose material, or trimming of slopes to a stable configuration with minimal use of retaining or protection structures.

A backhoe operating from quarry benches and temporary fill ramps is a most flexible tool for excavating and trimming batter slopes. On larger sites the development of a sluicing system may prove a viable option for clearing loose gravelly colluvium off steep rock faces.

Backhoe excavations will continually uncover more information on subsurface conditions, so ongoing supervision by the engineering geologist will enable rapid modification of the remedial programme when (or if) unexpected conditions are encountered. In the case of the deeply fractured 'massive lava' on Lot 5 (Balmoral Partnership) a quick inspection would have revealed that continued excavation would bring down the northern part of the Lot 24 bench, and if the quarry had not already been subdivided then the above course of action would be acceptable (and certainly cheaper than the gabion wall option).

Extensive excavations and trimming of batter slopes over the entire site would modify the original bench and slope topography on which the existing lot subdivision is based, but this would not be a problem if the lot boundaries were adopted after remedial works had modified the topography. The "one-hit" approach to investigations and remedial works (outlined above), followed by siting of lot boundaries and the inclusion of costs into purchase prices, would be a more cost and time efficient process than the piece-meal approach that had to be adopted at McCormacks Bay Quarry.

## 2.7 SUMMARY.

(1) Preliminary site mapping onto 1:200 scale plans drawn from limited topographic surveys, was sufficient to delineate areas that would require remedial works on former quarry batter slopes.

(2) Bedrock units, which include; air fall (ash), debris flow (lahar), and subdivided lava flow ('massive', 'vesicular', 'rubbly lava') units, occur in generally planer layers dipping off the volcanic cone. In detail deposition of units was controlled by pre-existing topography and emplacement mechanisms to give unpredictable variation of thickness on the 5 to 10m outcrop scale.

(3) Blocky fill, derived from quarry spoil, creates a compact stable platform for development of residential dwellings.

(4) Major risk to development of the site was from potentially unstable bedrock and colluvium on the quarry batter slopes. Quarry blasting practices and fracture style in the different bedrock units are the main controlling factors on areas of unstable rock.

The presence of cooling joints and a tendency to shatter when "over-blasted" make 'massive lava' the most fractured and potentially unstable rock unit on the batter slopes. Heavy blasting of 'vesicular' and 'rubbly lava' can result in open fractures and cavities defining large (5 to 10m$^3$)detached blocks, e.g. on southern part of Lot 24 batter slope.

(5) Field observations on the extent of fracturing, size and spacing

of blast holes, and sub-surface cratering at the base of blast holes have shown that the "heading blast", or "bulling" method was employed on McCormacks Bay Quarry. Difficult drilling conditions and the requirement to produce large blocks for rip-rap were probably the main reasons for continued use of this blasting method, which was generally superceded by full-face, or long hole methods in the 1950s.

(6) Remedial measures on the quarry batter slopes utilised a qualitative approach to stability assessment (such as that advocated by Grigg & Wong, 1987) and a "design-as-you-go" philosophy (Fookes & Sweeney, 1976) involving close co-operation between engineering geologist, certifying engineer and contractor.

Remedial measures used include: (a) removal of isolated loose blocks by hand, (b) clearing of fractured material and trimming of slopes by backhoe, (c) non-pressurised grouting of open fractures and cavities, (d) buttressing of large blocks with steel-reinforced concrete, (e) reinforced concrete block and gabion basket retaining walls, (f) tree and shrub planting on slopes to stabilise loose gravelly colluvium.

(7) Site investigations and remedial works at McCormacks Bay Quarry would have been simpler and less expensive if carried out as a single project before the fixing of lot boundaries and the sale of lots to individuals.

# CHAPTER 3 : COLERIDGE TCE LOESS BANK.

## 3.1 INTRODUCTION.

### 3.1.1 Site Description.

Coleridge Tce is a short (150m) NE-SW trending street at approximately 50m above sea level in the port town of Lyttleton, on the southern face of the Port Hills (Figure 3.1, location). The northern side of Coleridge Tce is bounded by a steep ($50^0$ to $90^0$) loess bank which cuts across a broad spur and ranges in height from 5m at either end to 8m in the central part (Figure 3.1, face log).

A small street (Coleridge Lane, Figure 3.1), that runs above the east half of the bank and protects most of the face from surface water run-off sourced from up-slope, but the Lane itself is threatened by erosion of the bank in the middle section (Figure 3.1, plan). The bench above the west of the bank is occupied by an unsealed footpath and a stone wall. Small slide/flow slope failures indicate the western end of the loess bank and footpath above, while the middle area of erosion truncates the footpath where it should join Coleridge Lane (Figure 3.1, plan). Vegetation consists of shrubs and creepers hanging over the loess bank below Coleridge Lane, with grasses and the occasional broom shrub growing on benches and gullies across the bank (Figure 3.2).

### 3.1.2 Objectives.

Site investigations were carried out at this site in order to produce an engineering geological model to aid in the design of an anchored retaining wall. The retaining wall, which is intended to prevent further erosion of the bank and protect against a possible major slope failure, is a Lyttleton Borough Council project involving a consulting engineer and earthmoving contractors. A memo from a meeting between the above parties and D.H. Bell (thesis supervisor) states: "7. Investigation of soils properties of the Coleridge Tce bank, presence of rock, shear strength, water distribution, localised seepage."

A more detailed set of objectives directed towards the "investigation of soils properties" for this thesis study are:

(1) to survey and draw up plans and face logs of the bank that allow detailed logging of the geomorphic features and loess layering.

(2) to log the geomorphic features and develop a model for site

Figure 3.2 : Coleridge Tce Loess Bank (east and west ends are not in view)

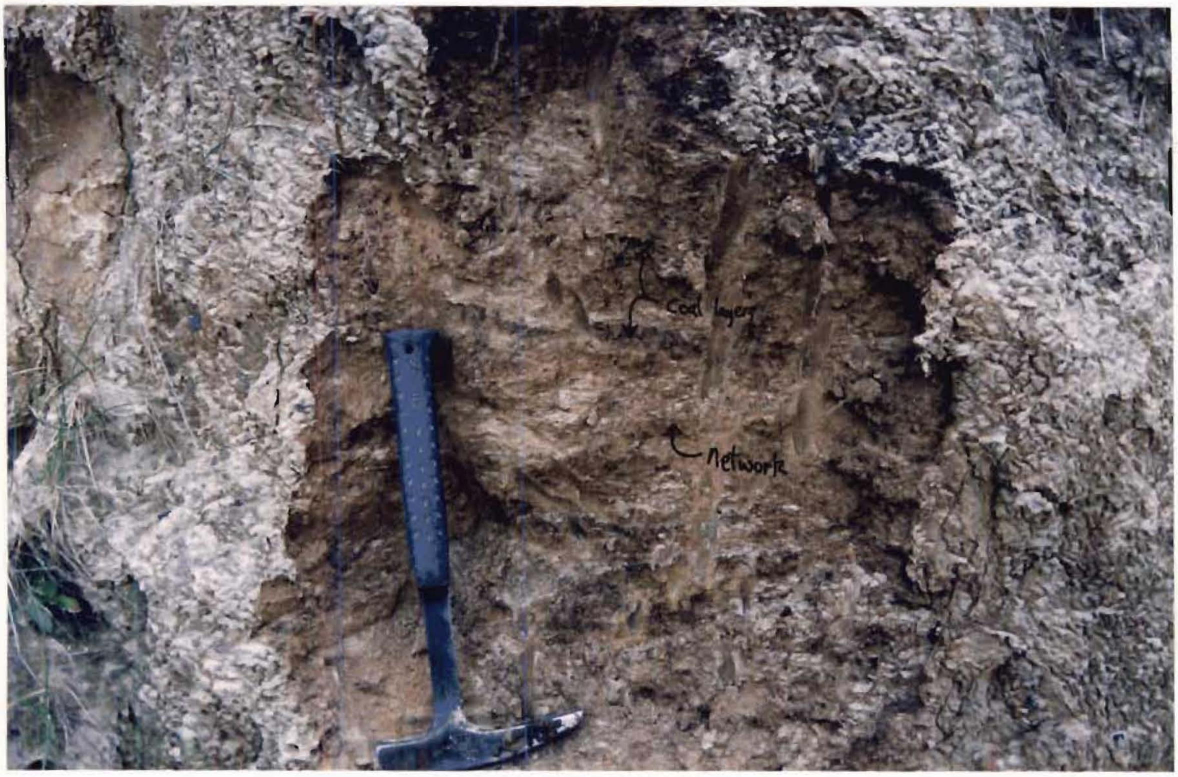



Figure 3.3 : Recent colluvium at the base of Coleridge Tce bank. Note network mottling and coal layers in cleared face, and slope wash silt lobes on surface

evolution in terms of erosional processes and rates.

(3) Determine the detailed loess layering exposed in the bank and attempt to quantify this by selective sampling and testing for index values.

(4) Carry out triaxial tests on samples at insitu moisture contents and higher moisture contents (close to the plastic limit) that could become established in the bank.

(5) Produce an engineering geological model of the bank (with index and strength parameters) that can be used in the design of the anchored retaining wall.

## 3.2 SITE DOCUMENTATION AND ENGINEERING GEOLOGICAL DESCRIPTION.

### 3.2.1 Survey and Logging.

The Coleridge Tce bank was surveyed in detail using an electronic theodolite and distomat. Surveying was carried out from an arbitrary base line set up along the south kerb of Coleridge Tce and was not tied into the Lyttelton cadastral network. One hundred points were fixed, mostly delineating changes in slope and erosion features on the bank. Eight pegs were placed at intervals along the face to allow location of sample sites and stratigraphic details without re-surveying.

Survey data was used to draw up 1:50 scale plans and face logs onto 16 A4 sheets of mm grid graph paper. The remaining geomorphological detail was measured by tape from the reference pegs and sketched onto the 1:50 scale sheets. Features delineated include: (a) positions of roads/paths above and below the bank, (b) scarps and benches across the bank, (c) erosional gullies, tunnels, chimneys (i.e. vertical tunnels) and semi-detached slabs,(d) slide scarps and debris piles, (e) areas of high soil moisture and sites of persistent overland flow or seepage. Completed plans and face logs, reduced to 1:100 scale and showing the features mentioned above, are given in figure 3.1.

The second phase of logging at Coleridge Tce involved determining the loess stratigraphy ( or loess layering within the bank) before starting a sampling programme for index testing and ultimately strength testing. Detailed inspection of the vertical succession in a mature loess bank, such as Coleridge Tce, is complicated by access to the vertical sections and covering of fresh exposure with a weathered zone, slope wash and vegetation. By comparison, subtle but potentially important details of loess layering are relatively easy to pick in freshly cut faces. Initial observations, gained by clambering over benches and using an extension ladder, resulted in a simple 4 layer model. This model was refined to a more detailed 'process related' sequence following backhoe excavation of channels in the bank to allow sampling of fresh material. Experience gained from logging fresh cuts at Westmorland (Chapter 4) helped in recognising and interpreting this detailed layering. Sections 3.2.3 and 3.2.4 discuss loess layering and interpretations.

### 3.2.2 Sub-surface Investigations.

Hand auger holes and single channel seismic refraction were considered as methods to test for the level of volcanic bedrock behind or below the face, and to ascertain the moisture distribution and loess layering behind the face. Seismic refraction was not used on this site for 4 main reasons: (a) the only place to run lines was along the road surface (above or below the bank), which creates a velocity inversion (seal and fill higher velocity than loess) and difficulties in interpreting depth to bedrock if a refractor was identified, (b) bedrock may be 10-15m or more below the surface, beyond the depth of strong signal penetration for a hammer energy source , (c) energy input for lines above the bank would be lost from the free surface of the bank, and (d) continuous traffic movements in the area would give a very high noise to signal ratio, probably limiting lines to 30m or less.

Hand auger holes have proved a useful tool in some loess investigations, penetrating to a maximum depth of about 3m, depending on density and moisture content of the loess (see Scott 1979, Yetton 1986, Tehrani 1988) . Hand auger holes drilled at Coleridge Tce penetrated to a maximum of 1.5m, drilling became very difficult with resistance to rotation at 1.5m threatening to break the auger , or to jam it in the hole. Results of the holes drilled are discussed in section 3.2.3. The need for information from power auger holes is discussed in section 3.3.1.

### 3.2.3. Loess and Bedrock Distribution.

A walk-over survey of the immediate area surrounding Coleridge Tce was carried out to search for evidence of volcanic outcrop and to establish the general geomorphic setting of the Coleridge Tce loess bank. The bank straddles a broad S-SE trending loess capped spur. Detail of the natural topography is obscured by suburban development of the area, but the spur would have originally been about 300m across (from gully to gully) and sloped at approximately $10^0$. No evidence of bedrock outcrop was found within about 100m of the bank, and although detailed logging of the bank showed up subtle details in the loess layering (discussed in section 3.2.4), no evidence of mixed colluvium or bedrock outcrop was found.

Hand-drilled auger holes to 1.5m below kerb level on Coleridge Tce (Figure 3.1, cross sections) showed no evidence of bedrock, or mixed (volcanic/loess) colluvium, which is often 0.5 to 2m thick overlying bedrock. The loess recovered from the auger holes was C1 type loess ( section 3.3.3) with moisture contents of 13 to 14 %. A maximum moisture content of 16% was obtained from the base of a hole drilled at the west end of the bank (see figure 3.1)

No definitive evidence exists on the proximity of bedrock to the Coleridge Tce bank, however some inferences can be drawn from the observations outlined above. The auger hole results suggest bedrock is at least 2m below Coleridge Tce, while the geomorphic evidence indicates it could be upto 10m below the Tce. The gently sloping spur is a remnant of a colluvially thickened wedge of loess that has been dissected by headwards erosion of streams. Bedrock base level of the streams may be as much as 25m below the crest of the spur (although details masked by residential development), therefore allowing for a minimum of topographic relief on bedrock the loess could be 20m thick at Coleridge Tce. Given a maximum thickness of loess of 20m, bedrock would be 10 to 12m below the kerb on Coleridge Tce.

Speculation aside, one of the most striking features of loess draping over the eroded Lyttelton volcano is the unpredictable bedrock topography and thickness of loess. In the absence of any other indicators, the presence of bedrock behind the loess bank must be tested by several vertical and inclined power auger holes drilled to at least 5m. Carefully logged and sampled auger holes will also serve the dual purpose of locating any significant loess layers, or changes in moisture content.

### 3.2.4 Loess layering.

Detailed layering in the Coleridge Tce loess bank was logged in the west and east backhoe channels cut for this purpose and to allow sampling of fresh, moist material. Measured sections of the channels, with detailed field descriptions, are produced on figure 3.1. Measurements are taken from the kerb at the base of the bank. The following paragraphs give loess layer thicknesses and interpretations of each layer for both channels.

West channel : (0-1.1m) brown grey silt - reworked buried soil and insitu loess, (1.1-1.95m) fragipan - coarsely mottled lower part of buried fragipan, (1.95-2.8m) brown grey silt with some mottling - reworked buried soil and insitu loess overprinted by upper layer of buried fragipan, (2.8-3.75m) yellow grey silt - 'network mottled' reworked insitu loess, (3.75-4.1m) brown grey silt - buried soil, (4.1-5.4m) yellow grey silt - reworked, 'network mottled' reworked insitu loess, (5.4-6.3m) fragipan - upper fragipan with prominant vertical fractures, (6.3-7.2m) yellow grey sandy silt - weathered/leached loess above upper fragipan,(7.2-7.3m) top soil.

East channel : (0-1.3m) fragipan - coarsely mottled lower part of buried fragipan, (1.3-2.25) brown grey silt with some mottling - reworked buried soil and insitu loess overprinted by upper layer of buried fragipan, (2.25-3.0m) yellow grey silt - network mottled reworked insitu loess, (3.0-3.2m) brown grey silt - buried soil, (3.2-4.1m) yellow grey silt - network mottled reworked insitu loess, (4.1-4.35m) brown grey silt - buried soil, (4.35-5.1m) yellow grey silt - network mottled reworked insitu loess, (5.1-5.4m) fragipan - coarsely mottled lower layer of upper fragipan, (5.4-5.85m) fragipan - vertically fractured upper layer of upper fragipan, (5.85-6.35m) yellow grey sandy silt - weathered/leached loess above fragipan, (6.35-6.5m) top soil, (6.5-7.3m) fill and slope debris from construction of Coleridge lane.

Three major types of loess layer are identified in the measured sections. They are yellow grey silt (network mottled reworked insitu loess), brown grey silt (buried soil) and fragipan (mottled clayey silt). The upper fragipan is approximately 1m thick and 5-6m above kerb level across the bank (see figure 4.1. plan, face log, sections). A buried fragipan and overlying gradational zone to yellow grey silt, with a total thickness of 2-2.5m, lies 1-3m above kerb level on the western half of the bank and 0-2m above kerb level on the eastern half of the bank. A single buried soil (brown grey silt) lies within the reworked insitu loess between the lower and upper fragipans in the west back hoe channel. Two thinner buried soils lie between the fragipans in the east back hoe channel. The soil horizon is assumed to split across the face (see figure 3.1 face log and sections). Field evidence and supporting index test values used in interpreting loess layers are outlined in the following paragraphs.

Yellow grey silt may be referred to as unaltered insitu loess. This is undifferentiated low clay (10-15%) loess that may be insitu airfall (discussed in section 4.4.2), or colluvially reworked networked mottled material. The essential factor is that the material has had negligible modification by soil/fragipan forming processes. Differentiating between airfall loess and reworked loess involves close inspection of fresh exposure to identify network mottling (reworked) or massive (insitu) structure and determination of dry density, which ranges from 1300-1500 $kgm^{-3}$ for airfall loess and 1500-1700 $kgm^{-3}$ for reworked loess. All yellow grey silt is highly erodible, with pinhole erosion values of E50 to E180. There is an increase in erosion resistance with increasing dry density. Yellow grey silt at Coleridge Tce is unlikely to be insitu airfall loess, because of the presence of network mottling, high dry density (1700 $kgm^{-3}$) and pinhole erosion values tending towards E180 (see table 3.1 and section 3.3.3)

Brown grey silt layers have the following features that indicate they are buried top soil layers : brown grey colour; relatively high clay (15-20%); rare rounded granules and pebbles of massive basaltic lava; evidence of intense worm burrowing; especially at the base of units into underlying yellow grey silt; large amounts of finely disseminated carbonaceous material indicated by strong reaction to peroxide treatment for grainsize analysis; layers may thin, split, thicken and reamalgamate in both down and cross slope directions (see section 4.4.2, Figure 4.5) . Brown grey silts have dry density of 1600-1800 $kg/m^3$, clay content 15-20% and pinhole erosion values of E360-1000.

Network mottling manifests itself as lenses of white to yellow grey silt 5 to 10mm across and 2 to 5mm high surrounded by a network of brown grey clayey silt. Where silt lenses are dominant the brown grey material looks like vertically compressed wire netting. If brown grey clayey silt is dominant there is more disturbance of the structure, possibly due to bioturbation.

Evidence for the mode of formation of network mottling can be seen in deposits at the base of the Coleridge Tce bank. Figure 3.3 shows network structure in unconsolidated material that includes layers of coal, clinker and rusted iron fragments, indicating the material is recent colluvium and slope wash. Figure 3.3 also shows the formation of network mottling, with slope wash forming small lobes, or lenses of silt that may be draped with clayey loess and organic rich soil. Repetition of the slope wash/draping sequence builds up a network mottled deposit. The gradation from network mottled yellow grey silt to brown grey silt is controlled by the increasing dominance of draping/soil forming processes over slope wash processes, along with increasing amounts of bioturbation.

A well developed fragipan features an upper compact layer of relatively high dry density (1700-1900 $kgm^{-3}$), relatively high clay content (15-20%) and a characteristic pattern of steeply dipping polygonal fractures. This is the hard band that stands out in weathered cut faces. The lower part of the fragipan has a similar dry density and clay content, but no persistent fracture pattern. It is characterised by abundant, coarse (upto 15mm) orange mottling and blue grey veining, or gammation. The upper and lower parts are both resistant to erosion (pinhole E>1000). Thickness may vary from 0.3-1.5m for each part, with total thickness seldom exceeding 2.0m. The upper, or most recent fragipan is usually identified by the hard band morphology and prominent vertical fractures, which are most obvious when the material is dry (i.e. 6-8% moisture). Buried fragipans are generally within the moist (approximately 13%) zone of the loess section and are recognised by the coarse orange mottling and gammation of the lower layer. A compact upper layer was not positively identified for buried fragipans at Coleridge Tce.

### 3.2.5 Erosional Processes.

The loess bank was cut to 0.5-1m behind the present kerb line during widening of Coleridge Tce approximately 40 years ago . Judging by the slope of the almost uneroded face at the east end and the original position of the footpath at the west end of the bank, it must have been cut at an angle of 80-90$^0$ from horizontal.

Four styles of erosion were identified during detailed geomorphic mapping of the bank. They are : (a) tunnel or chimney development along vertical fractures in the recent fragipan and underlying reworked insitu loess, (b) sliding or toppling of slabs of loess isolated from the face by collapsed chimneys and open fractures, (c) open gully and slope wash erosion by surface water runoff, (d) shallow seated (<2m) slide/flow slope failures in slope debris, buried fragipan and reworked loess/buried soil on the lower 2-3m of the bank.

Chimney development and slab failure are occurring in the dry, openly fractured recent fragipan and highly erodible reworked loess. Active chimneys occur on the bank above peg2 and 4 (Figure 3.1 face log) and are open to a maximum of 1m behind the face. Potential for chimney development is greatest where the surface above the bank is not protected by sealed road or thick vegetation (see position of Coleridge Lane and thick vegetation on Figure 3.1 plan and face log) and where the gently dipping recent fragipan is truncated at the surface (Figure 3.1 sections).

The enlargement of chimneys and fractures to isolate slabs of loess and the subsequent failure of slabs has lead to the distinctive bench and scarp morphology of the bank (Figure 3.1). Slabs may fail by toppling or sliding at the base. It appears that both mechanisms may operate as benches sloping at about $20^0$ and about $60^0$ are found. Benches of $20^0$ slope may be formed by toppling blocks failing in tension, while benches with $60^0$ dip may be formed by slabs failing in shear (note: $\phi=30^0$ and $45^0+\phi/2=60^0$, i.e. the benches are $60^0$ slide planes). A lack of recent slab failures limits the ability to more precisely model the failure mode, moisture conditions, loess layering, and slab size and shape.

Surface run-off has formed the narrow gully at the east end and is responsible for the most recent erosion at the west end and center of the bank (Figure 3.1 plan and face log). Open gully (and chimney) erosion at Coleridge Tce is much less severe than erosion by similar processes at many other cut loess banks on the Port Hills (see figure 3.4). This is because of the protection against surface run-off from up slope by the sealed road (Coleridge Lane), thick vegetation (east end bank) and established garden behind the block wall (west end bank).

Small slide/flow failures ($<5m^3$) have modified slope debris accumulated at the west end and center of the bank (Figure 3.1 plan). The larger arcuate features at the west end and center of the bank are probably larger slope failures, but much of the evidence , in the form of toe deposits, has not been preserved (removed to clear road). Slide/flow failures have only occurred on the lower parts of the bank, which generally have higher moisture content, and at sites where surface runoff has been persistent, i.e. at the center of the bank, from the end of Coleridge lane and at the west end, from the property behind the block wall.

## 3.3 SAMPLING AND INDEX TESTING.

### 3.3.1 Sampling.

Sampling for index tests and strength tests involved collection of disturbed bulk samples and undisturbed samples by driving and extraction of 35mm diameter thin walled stainless steel tubes of 50mm and 150mm length. Sample sites had to meet the following criteria: (a) sample loess layer of interest, (b) be accessible for clearing of face and handling of tube sampling equipment, (c) fresh exposure free of recent weathering effects and root penetration should be relatively close to the surface of the bank, so a minimum amount of excavation is required.

A dry compact loess face, like Coleridge Tce, can be very difficult to dig into beyond about 0.3m without the aid of mechanical devices (e.g.

Figure 3.4 : Eroded loess bank Lyttelton tunnel entrance. Compare with Figure 3.2


Figure 3.5 : Backhoe cutting sampling channel at West end of bank. Note loess is very hard and backhoe is tipping backwards, rather than cutting into bank

backhoe, jackhammer), so the need for a thin weathered layer (less than 0.3m) and an accessible platform to dig from place restrictions on potential sample sites across the face. The dry nature of exposed parts of the face restricts their suitability for sampling, especially for collecting tube samples. Dry loess (i.e. <8% moisture by weight, see section 3.3.3) is very hard, requiring 20-100 blows with a heavy driver to drive a 150mm tube home, and subsequent removal of the tube may require excavation of the full length. Putting aside the large amount of time and energy expended to gain one sample, the major problem is disturbance of the sample by the high number of driving blows. Many samples collected from dry loess were found to be crumbly and fractured in the tube and unsuitable for testing. By comparison tube sampling in moist (10-14% moisture) loess requires 5-20 driving blows resulting in minimal sample disturbance and easier removal of tubes from the face. Favoured sampling sites of higher moisture content occur at the base of the bank, beside gullies and at the back of grass covered benches.

Another problem with sampling, specifically tube samples for triaxial testing, is the presence of steeply dipping fractures parallel to and at high angles (50-70$^0$) to the surface of the face. These fractures are an exfoliation or slab weathering phenomena on the exposed face and normally do not penetrate beyond 0.5m into the face. Tube samples cut by fractures may be unsuitable for triaxial testing of material strength (section 3.4.2).

Initial samples were collected from suitable sites across the face, but before beginning a strength testing programme it was decided to cut two backhoe channels down the face to expose fresh, and hopefully moist material for sampling. The loess proved to be extremely hard, with little increase in the moisture content, even upto 1m into the face. Figure 3.5 shows a backhoe digging a channel at the west end of the bank. Detailed loess layering was logged and a suite of index samples collected from these layers. Tube samples for triaxial testing were collected from two horizons after the significance of the detailed layering had been assessed.

### 3.3.2 Index Test Methods.

Laboratory tests on loess material from Coleridge Tce are similar to those carried out on samples from Westmorland (see section 4.4) Index tests included grainsize determination (hydrometer method), Atterberg limits and moisture content, which were all carried out according to New

Zealand standards (Appendix 2.1). Pinhole erosion and crumb tests are modifications from original tests. The methods are outlined in Appendices 2.2 and 2.3.

Uniaxial swelling is another simple and potentially useful index test for loess, but it was not used because of the unavailability of equipment.

3.3.3 Results and Trends.

Results of index tests on samples from Coleridge Tce are given in table 3.1. Samples L1, L2 & L3 were the first samples collected after initial logging of the bank, and L4, L5, L6 & L7 were bulk samples collected to establish the range of moisture content across the bank prior to presenting some preliminary findings to the design engineer. Results from L1, L2 & L3 were considered suspect, so they were resampled and tested at a later date. C1, C2, C3 & C4 samples were taken from the west backhoe channel (see section 3.3.1) and C5, C6, C7 & C8 from the east backhoe channel. Sample sites are shown on figure 3.1 (face log), with C1 to C8 samples labeled Col1 to Col8.

Before discussing the data in table 3.1, it is worthwhile considering an observation from section 3.2.4 that helps explain the scattered nature of much of the index data. The observation is that loess layering is related to two factors, the amount of colluvial reworking and the degree of soil formation and related weathering/leaching. Colluvial reworking leads to mixing of layers and potentially angular and discontinuous contacts between layers. Soil formation and fragipan development is an incipient process, so boundaries between layers are often gradational. The ambiguity of contacts and incipient development of some layers can lead to the collection of unrepresentative samples, which results in a scatter of index values about a representative value for a given loess layer. This scatter can be great enough to create an overlap of values for materials from supposedly different layers. The only way to resolve the problems of overlaping values is to collect a statistical number of samples for each layer (e.g. 20 plus samples rather than 3 or 4) and average out the results.

The following observations can be made from table 3.1.

(a) Dry density is generally high, ranging from 1680 kg/m$^3$ to 1905 kg/m$^3$. Buried fragipans can be grouped around 1900, buried soils around 1850 and insitu/reworked loess at 1700 kg/m$^3$.

| Sample | Dry Density (kg/m³) | Insitu W % | Atterberg Limits | | | Grain size (%) | | | Crumb (class) | Pinhole Erosion | Loess layer |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | WL | WP | WI | clay | silt | sand | | | |
| L1 | 1780 | 7.2 | 26 | 18 | 8 | 19 | 70 | 11 | 3 | E>1000 | Upper fragipan |
| L2 | 1720 | 6.1 | 24 | 20 | 4 | 10 | 79 | 11 | 2 | E50 to E180 | Insitu loess |
| L3 | 1800 | 8.4 | 25 | 18 | 7 | 18 | 71 | 11 | 3/2 | E360 to E1000 | Buried fragipan |
| L4 | – | 6.7 | – | – | – | – | – | – | 2 | – | – |
| L5 | – | 11.8 | – | – | – | – | – | – | 2/3 | – | – |
| L6 | – | 14.3 | – | – | – | – | – | – | 3 | – | – |
| L7 | – | 14.4 | – | – | – | – | – | – | 3 | – | – |
| C1 | 1680 | 14.8 | 25 | 20 | 5 | 14 | 74 | 12 | 1 | E360 to E1000 | Buried Soil |
| C2 | 1880 | 12.0 | 24 | 16 | 8 | 19 | 69 | 12 | 3 | E>1000 | Buried fragipan |
| C3 | 1750 | 8.4 | 25 | 19 | 6 | 14 | 75 | 11 | 2 | E180 | Insitu loess |
| C4 | 1860 | 10.2 | 25 | 17 | 8 | 19 | 69 | 12 | 2 | E360 to E1000 | Buried soil |
| C5 | 1901 | 9.4 | 24 | 16 | 8 | 19 | 69 | 12 | 2 | E>1000 | Buried fragipan |
| C6 | 1780 | 12.1 | 27 | 17 | 10 | 18 | 70 | 12 | 3 | E360 to E1000 | Buried soil |
| C7 | 1780 | 9.2 | 21 | 16 | 5 | 19 | 70 | 11 | 3 | E50 to E180 | Insitu loess |
| C8 | 1850 | 10.9 | 25 | 18 | 7 | 19 | 69 | 12 | 2 | E360 | Buried soil |

Table 3.1 : Index Test Results for Coleridge Tce

(b) Insitu moisture content ranges from dry (6-8%), through moist (10-12%), to very moist (14-15%). Samples collected on the upper part of the face are dry, samples from the lower parts and behind grassed benches are moist, while samples from the base of the bank (where runoff and/or seepage are more persistent) are very moist.

(c) Atterberg limits show little variation, which is in line with minimal variations in grainsize and the absence of significant amounts of swelling clays in any samples. Two poorly defined groups of samples exist, one with plasticity index (WI) of 4-6 (low clay%), the other with WI 7-10 (higher clay%).

(d) Grainsize analysis shows a dominantly silt sized material, with the clay fraction varying from 10% to 19%. The grading curves, given in Appendix 7.1, fit into the classic wind transported silt (i.e. loess) grading envelope. Fragipan and buried soil materials have higher clay (19%), while insitu reworked loess has lower clay (10-14%). However the distinction is not totally clear and some apparently insitu loess layers have higher clay % (and dry density), which suggests reworking of layers may have been occurring within loess units. (Note that a loess unit is the material from above a buried fragipan, upto and including the next fragipan (buried or upper) and that reworking, as mentioned above, was occurring during the period of loess deposition, but before the development of the overlying fragipan.).

(e) The Crumb test, which is a modified version of an original test (outlined in Appendix 2.2), is a simple measure of the amount of dispersive clays present in a soil. Class 1 is no reaction, class 4 is a strong reaction of dispersive clays. Coleridge Tce samples are all class 2 or 3, with no obvious pattern or grouping of samples.

(f) Pinhole erosion is another modified test (see Appendix 2.3) that gives a relative measure of erodibility of materials from different loess layers. This is the one index test that shows variation in line with the layers logged in the face. Insitu reworked loess has E50-180, buried soils and reworked soils have E360-1000 and buried (and upper) fragipans have E>1000. E50 is highly erodable and E>1000 is very resistant to erosion. The numbers 50, 180, 360, 1000 refer to mm heads of water used in the test.

(g) The loess type/layer interpretations for each sample are based on detailed logging of the exposure, broader observation of field relationships (at Coleridge Tce and Westmorland) and the index test data in table 3.1 (also index data from Westmorland, see section 4.4).

Interpretations of loess layering were discussed in section 3.2.4.

As mentioned in section 3.3.2, samples were not tested for uniaxial swelling. Yetton (1986) made a study of uniaxial swelling while investigating erosion mechanisms involved in tunnel gully formation. He concludes that rapid (few hours) expansion from air dry, with upto 20% strain indicates expansion by slaking and is highest in non-plastic silt/sand material. Slow (days) expansion to approximately 10% strain indicates dominantly clay swelling and occurs in the compact layer (fragipan).

## 3.4 TRIAXIAL STRENGTH TESTING.

### 3.4.1 Test Method.

The only strength test used on the loess material was an unconsolidated, undrained (UU) triaxial test on partially saturated 35mm diameter tube samples (see Appendix 3.1 for method) . Direct shear tests and other types of triaxial test were not carried out in this study. Reasons for not using other test methods are outlined in Appendix 3.2.

The quick UU triaxial test allowed for the relatively rapid processing of many samples. Suites of samples could be tested at commonly encountered field moisture contents, as well as elevated moistures likely to occur in natural slope failures, i.e. around 18-20% moisture and higher (at or above the plastic limit). Simple methods were used to try and elevate moisture contents in samples prior to testing. These included standing heads of water over samples still in the tube and moist curing extruded and trimmed samples in a fog room. Soaking and curing times upto 2 weeks were used.

### 3.4.2 Results.

52 150mm long tube samples were collected during 5 visits to the Coleridge Tce bank, and of these samples 41 were successfully extruded and loaded into the triaxial testing apparatus. This high success rate is due to very carefull sample collection and handling, e.g. collecting 10 tube samples required about 4 hours on site. Early attempts at collecting tube samples from Coleridge Tce (and Westmorland) resulted in less than half the samples being suitable for testing. From the 41 samples tested, 32 could be grouped into 7 suites, with a minimum of 4 samples per suite. The suites are divided into 2 loess layer types sampled for index testing

(C1 and C2, see section 3.3), with individual suites defined by moisture content limits.

Data from the quick UU tests were interpreted in the following way: (a) the failure point for a sample was taken as the peak of the load/displacement plots, or at 10% strain for the samples that deformed plastically, (b) the principal stress ($\sigma_1$) and confining pressure ($\sigma_3$) values were converted to P,Q values and plotted on P/Q axis, (c) a $k_f$ line was fitted, by eye, through the data points, (d) total stress parameters (c,$\phi$) were calculated from the gradient of the $k_f$ line.

Adoption of 10% strain as failure for plastically deforming samples follows the idea of Kie (1988) concerning acceptable levels of soil deformation when designing engineering structures. Fitting a straight line to P,Q data by eye is recommended by Fell & Jeffery (1987) to allow for data variables not modeled by linear regression curve fitting. Plotting Mohr circles and line fitting to P,Q plots are compared for Westmorland triaxial data (Appendix 4.3 ). The relationship between a,$\alpha$values ($k_f$ line) and c,$\phi$ are defined in Millar (1982) and Fell & Jeffery (1987), see appendix 4.1.

Table 3.2 sets out the strength data and relevant index values for the 7 suites of samples. Detailed data and P,Q plots for each suite are given in appendix 4.1. Each set of c,$\phi$ results are total stress parameters for a given moisture content and range of confining pressures ($\sigma_3$).

The following are the major trends observed from the results: (a) cohesion (c) decreases with increasing moisture content, (b) angle of friction ($\phi$) appears independent of moisture content, (c) C2 suites (lower void ratio and clay content) appear to have a higher angle of friction than C1 samples, by approximately $3^0$, (d) values of cohesion are similar for C1 and C2 suites of similar moisture content.

### 3.4.3 Discussion.

This section further discusses and attempts to interpret the shear strength data. Data and conclusions are compared with 3 papers that gave strength data for loess from China, Hungary and the Arabian Peninsula.

Lambe and Whitman (1979) give a theoretical failure envelope for UU test on partially saturated materials (see figure 3.6). The envelope is curved concave downwards, trending towards a $\phi = 0^0$ line as increasing confining pressure increases the degree of saturation (S). C,$\phi$ parameters for Coleridge Tce samples are straight line approximations for the low confining pressure part of this curve (see dotted line on figure 3.6)

| Loess Layer | Clay (%) | Void Ratio (e) | 9 | 11.5 | 12.5 | 15.5 | 19 | W % (±0.5) |
|---|---|---|---|---|---|---|---|---|
| Reworked buried soil C1 | 14 | 0.588 | – | – | 40, 32 | 25, 30 | 0, 30 | total stress c, ø (kPa, °) |
| | | | – | – | 50-175 | 50-150 | 50-200 | range $\sigma_3$ (kPa) |
| Buried fragipan C2 | 19 | 0.395 | 140, 33 | 90, 34 | 40, 36 | 20, 32 | – | total stress c, ø (kPa, °) |
| | | | 50-150 | 50-200 | 50-200 | 50-200 | – | range $\sigma_3$ (kPa) |

Table 3.2 : Strength Data from Triaxial Tests of Coleridge Tce Loess

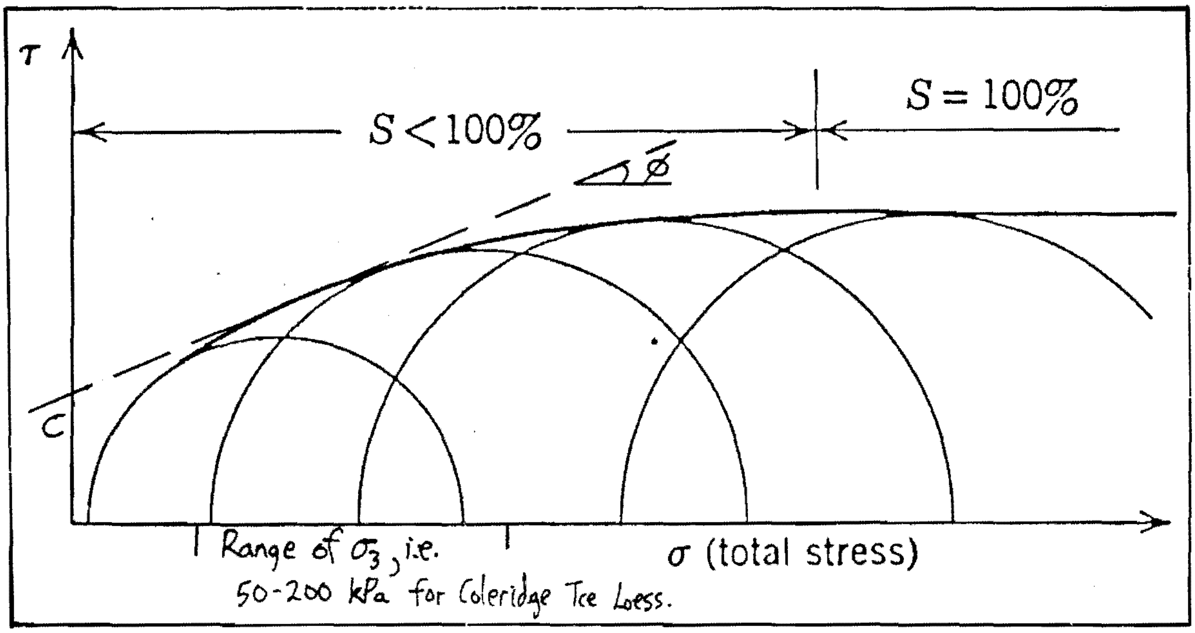



Figure 3.6 : Theoretical failure envelope for UU tests on partially saturated soil (after Lambe and Whitman, 1979). Dotted line represents segment of curve approximated by strength parameters for Coleridge Tce loess

Results in table 3.2 are undrained strengths at specific moisture contents, and for a limited range of confining pressures. The range of confining pressures (50-200 kPa) represent a typical range of overburden pressures found in thick loess (upto 15m+) on the Port Hills.

Graphing cohesion and friction angle data against moisture content highlights the trends noted in section 3.4.2, i.e. c decreases with increasing moisture content and $\phi$ is unaffected by increasing moisture. An added feature to note is a change in slope of the cohesion moisture content curve at about 12-13% moisture (see figure 3.7). The gradient of the line changes markedly, suggesting that this moisture content may coincide to a major change in loess behaviour. Cohesion increases very rapidly for moisture contents less than 12%, compared to cohesion increase between 19% and 12%. Three main properties have been forwarded as controlling factors for cohesion in loess. They are calcite cementation, clay bridge bonding and soil suction by capillary effects in partially saturated material (Erol & El-Ruwaih, 1982; Kie, 1988; Barden etal, 1973). A possible explanation for the observed cohesion variation (figure 3.5) is that soil suction provides the very high cohesion values for moisture contents of about 6-8%, with the influence of suction decreasing rapidly towards 12-13% moisture as open connections between pore spaces decrease. From 12% to 19% moisture there are few interconnected pore air spaces and little suction effect. Cohesion decrease from 12 to 19% moisture may be primarily due to softening of clay bridge bonds. Calcite is not present in large enough concentrations to have any effect on material bonding.

Published papers dealing specifically with the shear strength of loess is rare. Three papers, that include sections on strength of loess, partly support and partly contradict the findings from Coleridge Tce samples. Erol & El-Ruwaih (1982) show effective strength parameters for dry and saturated (no specific moisture contents) Arabian Peninsula loess tested by direct shear. They conclude that the friction angles ($\phi'$) of $26^0$ and $24^0$ show no significant change with increasing moisture content, but the decrease in cohesion (c′) from 46kPa to 10kPa is strongly influenced by increasing moisture content.

Kie (1988) discusses triaxial tests on Chinese loess, but unfortunately is unclear on methods and results. Two results can be gleaned from this paper : (1) c,$\phi$ values of 110kPa, $32^0$ where obtained for loess at 10.2% moisture and confining pressures of 0-200kPa, (2) triaxial tests, of an unspecified nature, run at moisture contents ranging from 8% to 20% gave friction angles from $17^0$ to $5^0$ (i.e. friction angle decreased with increasing moisture content.

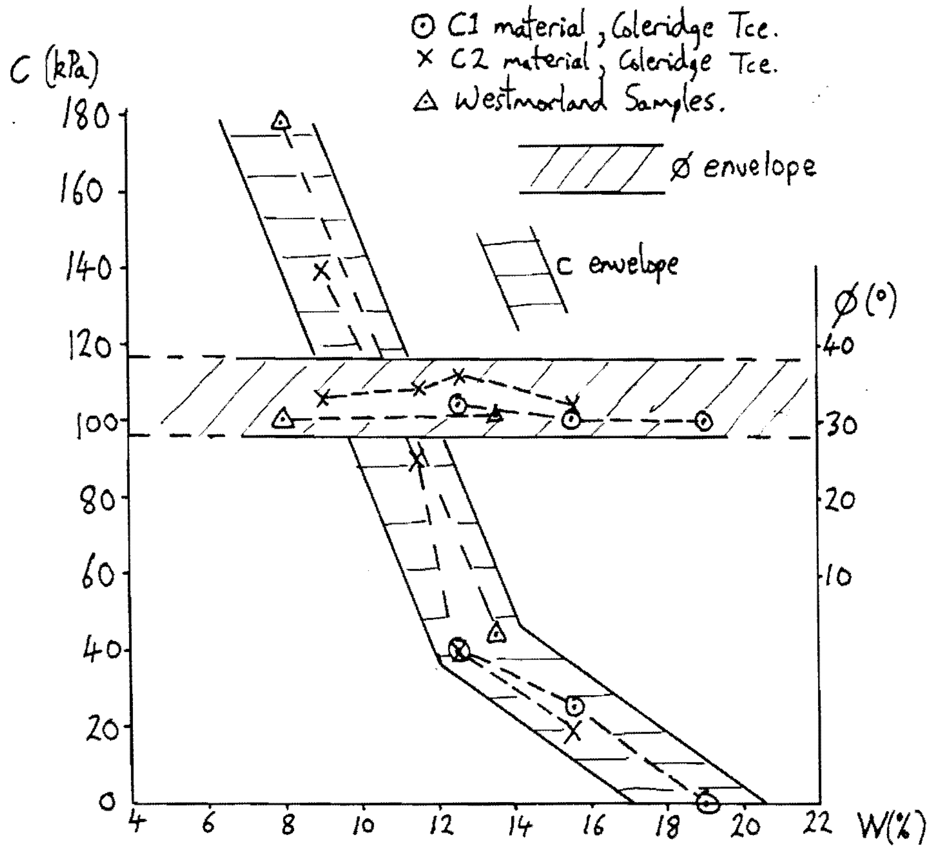



Figure 3.7 : Cohesion (c) and friction angle (Ø) trends for UU tests on Port Hills loess

Triaxial and direct shear tests by Pardanyi & Vago (1973) on Hungarian loess yielded 2 major conclusions. The first finding is that there is no significant difference between results from triaxial and direct shear tests, and the second is that both cohesion and friction angle decrease with increasing moisture content and increasing void ratio. Coleridge Tce results do not show a decrease in friction angle with increasing moisture content, but they may show increasing friction angle with decreasing void ratio, i.e. C1 e=0.588, av$\phi$=31$^0$; C2 e=0.395, av$\phi$=34$^0$.

### 3.4.4 Conclusion and Recommendation for Further Work.

Quick UU triaxial tests of samples from Coleridge Tce bank have provided useful information in terms of values of strength parameters and a general indication of the behaviour of loess under varying moisture conditions. However the sampling and testing programme was limited in scope, so results should be treated with some caution.

The results and comparisons with published data do allow preliminary conclusions to be drawn, with the most important being that shear strength of loess is primarily governed by its dominantly silt grainsize, density and moisture content. This is an important conclusion, because it suggests that a detailed study on typical Port Hills loess material would yield results applicable to all Port Hills loess sites - and, with a minimum of correlation tests, applicable to New Zealand loess in general. A detailed study should cover the following recommendations for further work.

(1) Investigate strength variations across the 10-14% moisture content range.

(2) Carry out more tests in the 18-22% (and above) moisture content range to test Coleridge Tce results that are based on limited data.

(3) Test samples at confining pressures in the 150-350kPa range, to try and define the change to a $\phi$=0$^0$ condition.

(4) Define c,$\phi$ values with more samples (10-15) per suite.

(5) Test finding that $\phi$ is independent of moisture content, as this goes against the findings of Pardanyi & Vago (1973) and Kie (1988).

(6) Investigation of mechanics of strength variations with varying moisture contents, e.g. clay bonds, soil suction.

## 3.5 ENGINEERING GEOLOGICAL SITE MODEL.

This site model draws on interpretations of loess layering, erosional processes and material characteristics from index and strength testing.

Figure 3.8 shows a soil profile model for use in designing a retaining wall and sketch sections of failure modes actively eroding the bank.

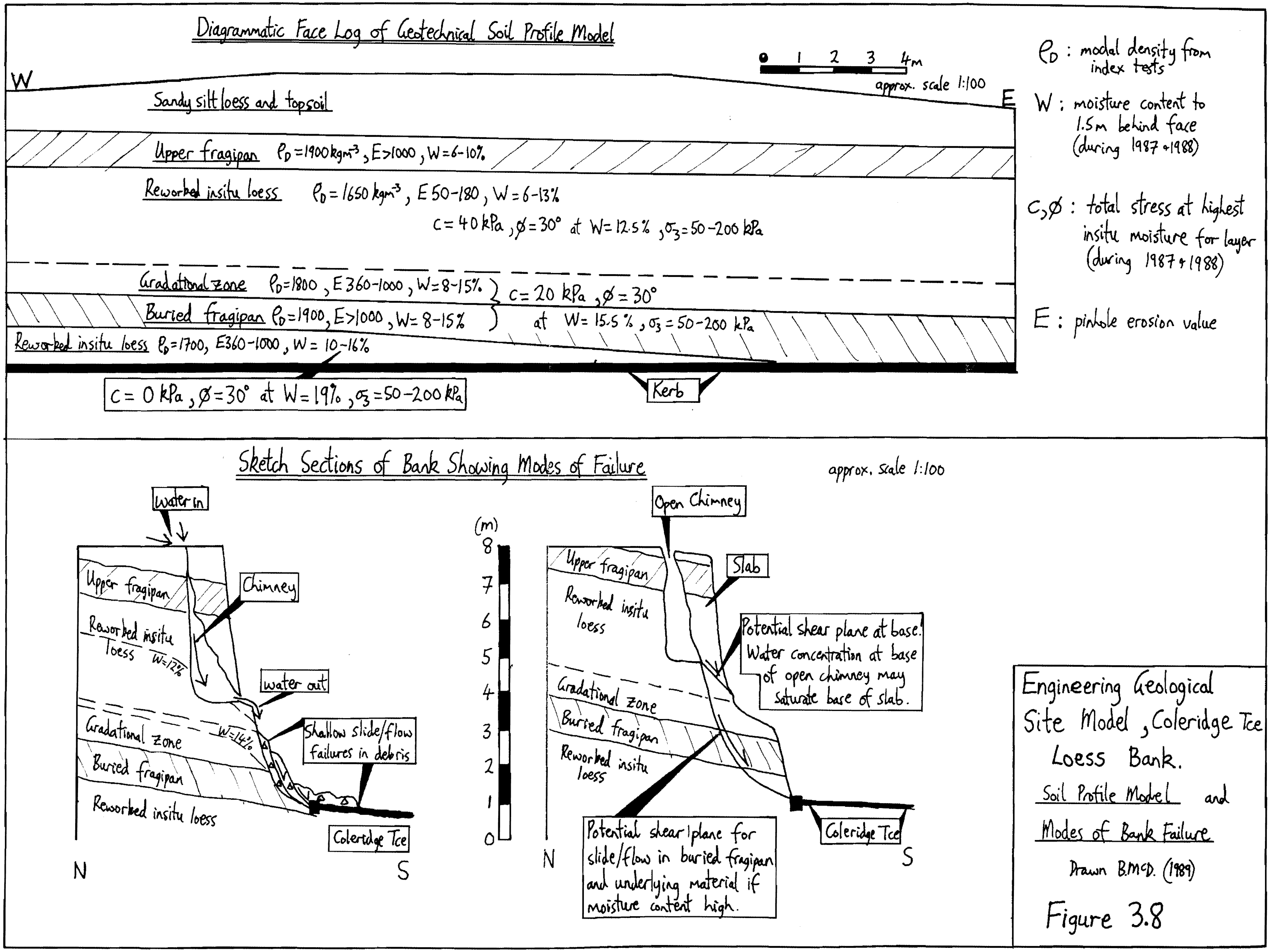
The soil profile model contains two main material types (fragipan and reworked/insitu loess), with a gradational zone (brown grey silt with some mottling) above the buried fragipan. Buried soil layers have been ignored in the model, as they have no effect on the style of erosion across the bank. Accurate positions of contacts between layers are plotted on the plans and sections of figure 3.1. Note that a gentle ($5-10^0$) southerly dip is inferred for the fragipan layers. Moisture contents given on the model are maximum and minimum values obtained from samples collected in 1987 & 1988, while strength parameters are at maximum measured insitu moisture for that layer.

Contour lines of equal moisture content are dotted onto the sketch sections to infer an insitu moisture distribution up to the free face. The site model suggests that surface run-off across the face and sub-surface flow within 2m of the free face (see sketch sections, figure 3.8) has had a greater influence on past stability of the face (see section 3.2.5) than a general rising of soil moisture within the loess profile. Prediction of soil moisture distribution in extreme (wet) conditions could be important in avoiding over design of a retaining structure. However this would require drilling of several holes, with sampling to establish insitu moisture at the time of drilling. Vertical drill holes into Coleridge lane could be monitored for soil moisture fluctuations with a nuclear probe, but this would need at least 1 years monitoring and some prolonged periods of wet weather to reach any conclusions on soil moisture distribution and temporal variations.

Two main factors stand out as essential for long term stability of the Coleridge Tce bank. They are maintainence of a drained condition in the core of the bank and control of surface water run-off, especially at the east and west end and the center of the bank where the greatest amount of slope failure/erosional activity has occurred. If surface water control and free drainage are incorporated in a wall design, then a retaining effect may only be necessary to hold thin slabs and debris piles against the face.

Having speculated on possible soil moisture conditions, it is worth noting that the cost of conservative design (i.e. raise soil moisture content throughout the face to reach a worst possible case and design a wall to fit) of a retaining wall for a small site, such as Coleridge Tce, is probably less than the cost of obtaining good data on soil moisture distribution and modelling the moisture fluctuations.

Research aimed at understanding soil moisture conditions in thick




Engineering Geological Site Model, Coleridge Tce Loess Bank.

Soil Profile Model and Modes of Bank Failure

Drawn B.McD. (1989)

Figure 3.8

loess deposits and steep faces in thick loess would require several years detailed monitoring of stable and unstable sites, but could yield useful information for economic design of future structures in loess. This type of research could be coordinated with the further work in shear strength outlined in section 3.4.4.

## 3.6 SUMMARY.

(1) Coleridge Tce cuts across a broad, gently dipping spur composed of loess 10-20m thick (section 3.2.3). The cut bank is 8m high, so drilling is needed to establish the actual depth to bedrock (i.e. 2-12m below Coleridge Tce).

(2) Three major loess layers are identified in insitu loess (section 3.2.4). Network mottled reworked insitu loess may be called yellow grey silt, and has relatively low clay%, low dry density and high erodibility.

Buried soil type loess, or brown grey silt is decomposed buried top soil, with medium to high clay%, medium to high dry density and medium erodibility.

An upper and buried fragipan were identified by prominent vertical cracking, coarse orange mottles and blue grey veining, and high clay%, high dry density, very low erodibility.

(3) Erosional processes (section 3.2.5) that have formed (and are forming) the morphology of the bank are : (a) chimney (vertical tunnel) development in the upper fragipan and insitu loess, (b) failure of slabs isolated by collapsed chimneys, (c) open gully and slope wash erosion by surface water runoff, (d) shallow seated (<2m) slide flow slope failures in slope debris, buried fragipan and reworked loess/buried soil on the lower 2-3m of the bank.

(4) Sampling for index and triaxial testing (section 3.3.1) was complicated by difficult access and the very hard dry nature of much of the bank. Sampling and detailed logging of the bank was aided by cutting two channels down the bank with a backhoe.

(5) Commonly used index tests (section 3.3.2 and 3.3.3) were carried out to test for variations in field identified loess layers, and to test the usefulness of the different index tests in loess investigations.

Crumb tests did not show any variations consistent with loess layering. Atterberg limits showed little, or no consistent variation between loess layers. Grainsize results showed broad grouping, but with considerable overlap for some loess layers. Dry density and pinhole erosion showed good grouping of results for different loess layers. Moisture content is an important test that quantifies moisture variations across the bank.

(6) A triaxial testing programme was carried out using 35mm diameter tube samples and the UU test method (section 3.4). Total stress parameters (c, $\phi$ ) were defined for buried fragipan (C2) and reworked buried soil/insitu loess (Cl) type materials at moisture contents from 8 to 19%. Cohesion (c) ranged from 180 to 0 kPa and internal friction angle ($\phi$) was 30 to34$^0$.

Friction angle was found to be independent of increasing moisture content. Cohesion decreased with increasing moisture content. The rate of cohesion decrease changes markedly at 12 to 13% moisture. Changing roles of soil suction and clay bridge bonding are envoked to explain strength behaviour across the 10 to 14% moisture range. A programme of further work is recommended (section 3.4.4) to test and expand on the above results.

(7) Aspects of loess layering, erosional processes and physical properties (index and triaxial results) are combined to produce a geotechnical soil profile model and erosional development model (modes of failure) for the bank (section 3.5, Figure 3.8).

The main conclusion is that stormwater and bank drainage management are crucial to the long term stability of the bank. Given adequate moisture/water control the retaining role of a wall would have to be minimal.

# CHAPTER 4 : WESTMORLAND SUBDIVISION EXTENSION.

## 4.1 INTRODUCTION.

### 4.1.1 Background.

The Penruddock Rise extension of Westmorland subdivision is located on the southern edge of a gently to moderately sloping (5-20$^0$) loess draped basalt spur above the existing development (Figures 4.1, 4.2 and 4.3). Proposals for development involved two areas of cut and two areas of fill along the road alignment. The maximum planned cut depth was 8m, with a maximum fill depth of 6m in a steep sided gully that dissects the loess mantle on the West side of Westmorland spur (Figures 4.1, 4.2, 4.3). The author was involved in preliminary site investigations followed by monitoring and documentation of the cut and fill operations. Engineering geology mapping along with some seismic refraction profiles and backhoe test pits were used to confirm the feasibility of the proposed road alignment and cut and fill operations.

### 4.1.2 Objectives.

The monitoring of this cut and fill operation has 3 major objectives in respect to this thesis. Site investigations, including engineering geological mapping and limited sub-surface investigations, were carried out to test the feasibility of the proposed earth works. Fill placement in the gully was monitored in terms of fill quality, and several density measuring techniques could be compared as part of the quality control. Lastly, the relatively extensive areas of fresh cuts were logged and sampled with the aim of describing layering in the thick (>10m) loess "sheet".

## 4.2 PRELIMINARY INVESTIGATIONS.

### 4.2.1 Engineering Geology Mapping.

#### 4.2.1.1 Introduction.

Conventional geological mapping is a limited tool at Westmorland. Even with large scale mapping the mappable units are limited to insitu loess, loess colluvium and volcanic bedrock. Volcanic rocks may be divisible

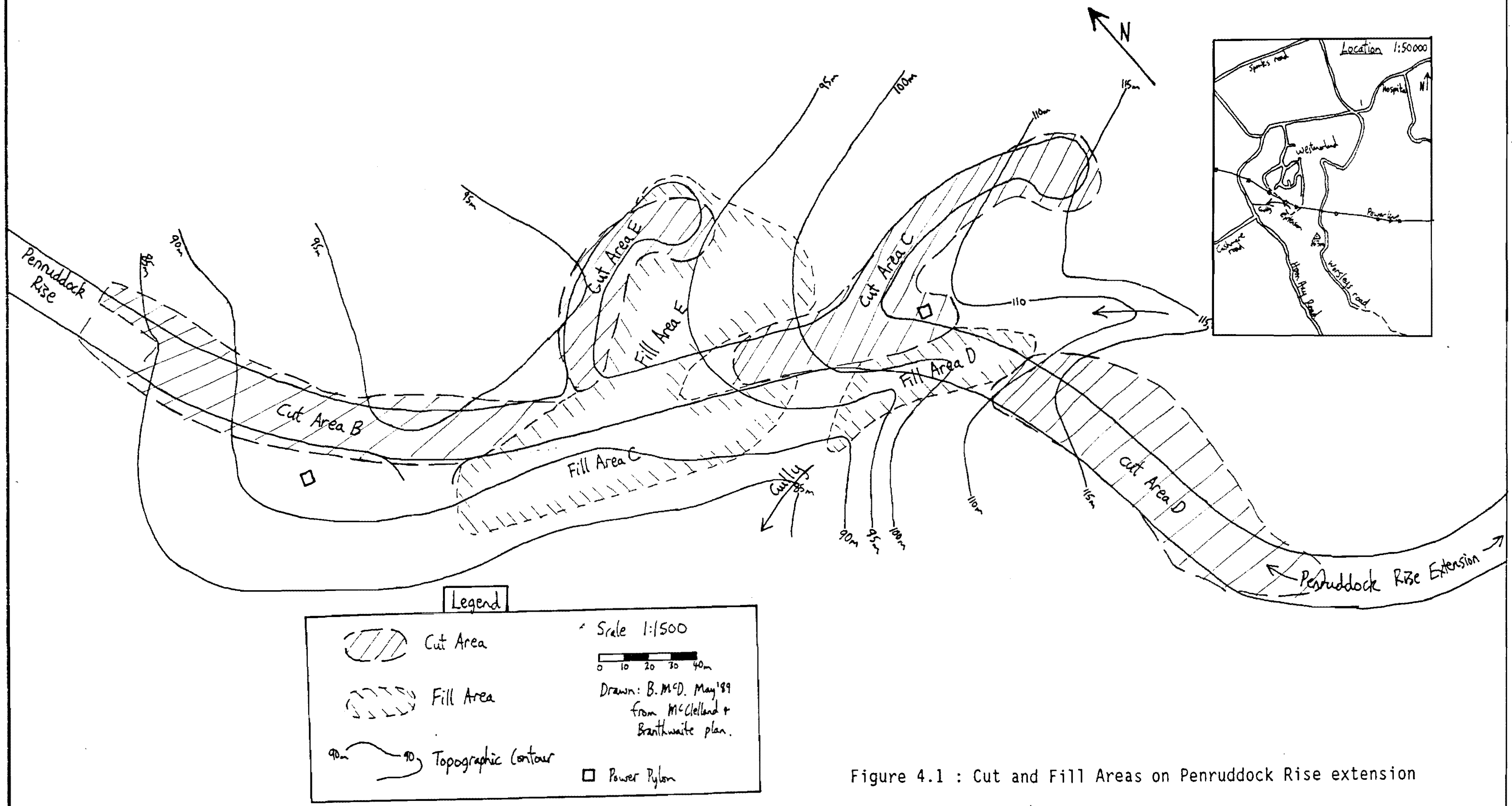



Figure 4.1 : Cut and Fill Areas on Penruddock Rise extension

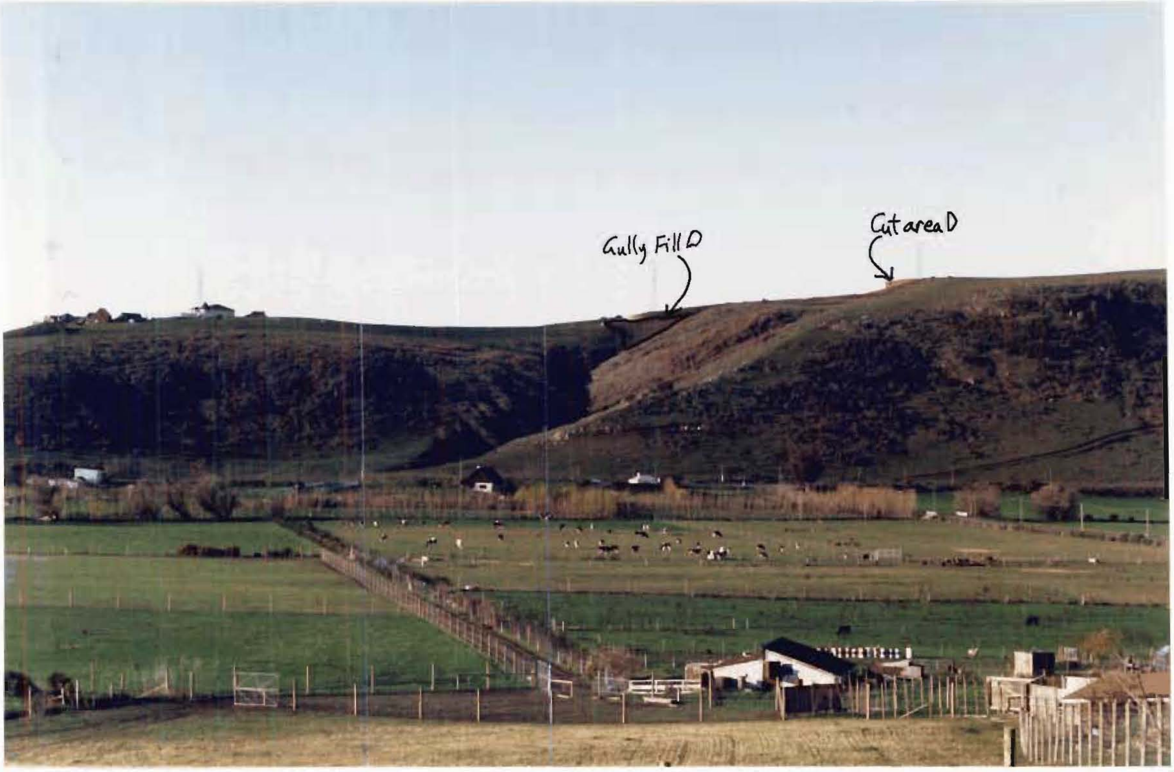



Figure 4.2 : View of SW side of Westmorland Spur. Note Gully and Gully Fill


Figure 4.3 : View Across Gully to Cut Area D and large Inactive Landslide Scarp (arrowed). Note fill area D and gullys in loess colluvium (arrowed)

into lava flows, pyroclastic deposits and dykes given extensive exposure. Layering within insitu loess is only recognisable in fresh cut faces, with the mantling nature of successive layers obscuring contacts in a map or plan view. The main objective of mapping is to record geomorphic features, especially those related to erosion of the loess mantle. 1:500 scale contoured topographic plans were available for the gully and cut and fill areas from the project civil engineers. The 1:500 sheets and low level air photos were used in a walkover survey of the entire length of the gully and slopes above and below areas of cut and fill. Basalt bedrock, insitu loess, loess colluvium, active and inactive erosion features were mapped.

#### 4.2.1.2 Description of Gully.

Refer to figure 4.4 (engineering geological map and sections) during the following description of geology and geomorphology. A NW dipping stack of basaltic lava flows (Lyttleton Group volcanics) is exposed on the steep SW bounding face of Westmorland spur (see figure 4.2). Thin (maximum of 3m) discontinuous sheets of mixed colluvium and loess colluvium obscure bedrock on the face.

The lower part of the filled gully (downstream of fill area D ) is incised into bedrock, with a covering of loess colluvium over the flanks or gully walls. Tunnel gully and rill erosion is very active in this loess colluvium, especially on the north facing slope. Exposures of basalt in the gully floor and high on the SE flank of the gully indicate that the bedrock surface parallels the slope angle on this face.

The middle part of the gully (fill area D) has eroded to a smooth sided shape within thick (10m+) insitu loess. Excavations in the gully floor for fill placement revealed 0.5-2m of bouldery mixed colluvium overlying weathered bedrock, so the gully has eroded to a bedrock base level.

Three straight line segments can be defined in the gully: (a) a lower part eroded into a pre-loess bedrock gully (W-SW trend), (b) a middle part (including fill area) eroded into loess to a bedrock base level (W trend), and (c) an active upper part of gully eroding headwards directly up the slope of the spur (NW trend).

#### 4.2.1.3 Gully Erosion.

Open gullies and partly collapsed tunnel gullies are restricted to loess colluvium on the flanks of the lower gully and the SW face of

Westmorland spur. Collapsed tunnels also occur on gentle slopes where loess is thin (<3m) and the tunnels drain onto steep bedrock/colluvium slopes(Figure 4.4) . No evidence for active tunnel gullying was found in the thick (>5m) insitu loess deposits around the upper part of the gully. Excavations for cut and fill along the road alignment revealed rare, small (0.3m diameter) tunnels developed above the upper fragipan.

4.2.1.4 Slope Instability.

Evidence for slope failure (slide/flow complexes) is found on gentle to moderate slopes on both sides of the gully, and on the steep slopes in the actively eroding upper part of the gully. A series of small (5-20$m^3$ of debris) slide/flow failures have occurred on the south flank of the active upper gully, filling the gully floor with debris. Intermittent surface water flow in the gully floor is actively eroding these debris lobes. Study of past air photos brackets the time of initiation of these features as between 1940 and 1961, with reactivation and enlargement during following wet periods. It is also notable (from air photos) that the major gullies and collapsed tunnels on Westmorland spur have shown negligible development in the last 40 years, apart from new fill areas.

Numerous small inactive slide/flow failures are expressed on the gentle to moderate slopes W-SW of the road alignment by 0.5-1m high scarplets and degraded debris lobes (approximately 5-10$m^3$). Scarplets are superimposed on a large arcuate scarp that is traversed by the road extension and cut area D (see figures 4.1, 4.3 and 4.4). This large scarp has the form of a slope failure, but the morphology of the face has been modified by superimposed small failures, and there is no obvious toe deposit. The area that could be occupied by toe debris is a gently sloping surface with a subdued hummocky topography (maximum relief 0.5m). Toe deposits could have been reworked over the edge of the lower gully, as loess colluvium over bedrock is thickest along this section of the gully flank.

There are 2 options for the origin of the large scarp: (a) a large slope failure in thick loess, or (b) a natural depositional feature. If the scarp was a depositional feature the upper fragipan would follow the contour of the slope, however excavations across cut area D showed that the upper fragipan and underlying buried soils (see figure 4.5) are almost flat lying and truncated by the scarp. The body of this evidence indicates a large slope failure (possibly related to erosion of loess infilling the pre-existing bedrock gully) that occurred after development of the upper

Figure 4.5 : Fresh Exposure in Cut Area D showing Multiple Buried Soil Horizons (arrowed) in Uppermost Loess. thin topsoil and mottled fragipan have been stripped from cut

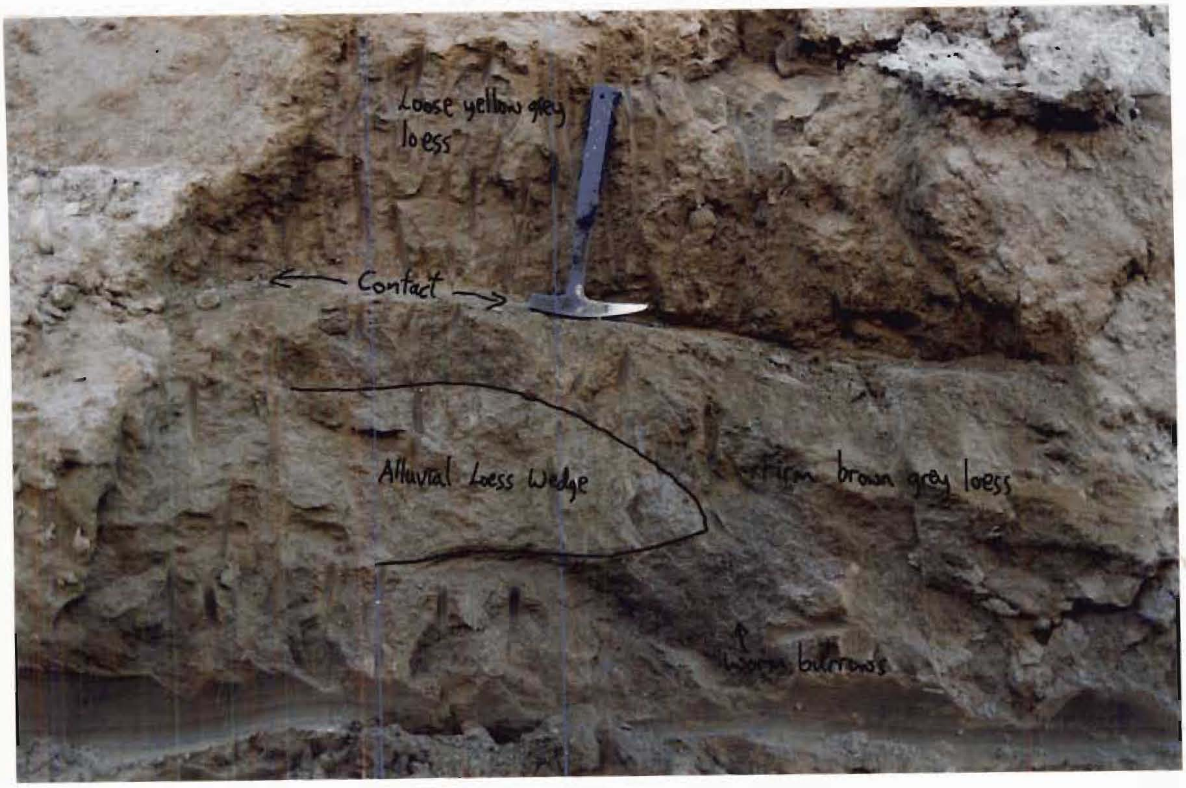



Figure 4.6 : Cut in South Flank of Gully showing Layering in Loess. Small alluvial wedge of sandy silt in relatively dense brown grey silt with worm burrows. Wavy, gently dipping contact with overlying loose massive yellow grey silt

fragipan, but far enough into the past to allow for removal of almost all the toe debris from the gentle slope (age range 500-5000 years ?).

4.2.1.5 Stability of Gully at Fill Area D.

An unusual feature of the middle part of the gully is the smooth side walls or flanks. Side walls of deeply incised, headwards eroding gullies are usually dissected by collapsed tunnels and open gullies aligned directly down slope.

Excavations across the middle part of the gully show a truncated gently dipping upper fragipan, pointing to post-fragipan headwards erosion of the gully. The excavations that revealed a truncated upper fragipan also showed a weakly developed younger fragipan and relatively well developed topsoil dipping parallel to the gully flanks. A relatively stable state is inferred for the middle part of the gully from the smooth flanks, weakly developed fragipan and top soil, and outcrop of bouldery colluvium and bedrock in the gully floor. The gully can erode no deeper and the flanks have established a stable configuration in the thick loess.

Erosion of this part of the gully may have been similar to the processes occurring in the active upper part of the gully, i.e. downcutting of a deep, narrow channel, with side wall development by slide flow failures.

4.2.2 Seismic Refraction Surveys.

4.2.2.1 Methodology.

Two refraction profiles were run across cut areas on the proposed road alignment (Figure 4.4), using a single channel signal enhancement (or hammer) seismograph. The purpose was to obtain a volcanic bedrock profile and to estimate loess thickness across areas where loess thickness could be less than the proposed cut depth. Cut area B (figure 4.1) is across a high point, or roll on the spur where wind speeds could be expected to be high, therefore decreasing the potential for loess deposition. Earlier backhoe pits east of the road alignment had struck bedrock at 1-3m below the surface (P.J. Kingsbury pers comm, 1988). Cut area D (see figure 4.1) crosses the large scarp inferred in the previous section to be a slope failure feature. A refraction profile was run across the scarp to test the possibility that the loess was draping across a major step in the underlying bedrock.

50m lines were set out along the road center line, and forward and reverse profiles were run to allow application of the reciprocal method of interpretation. Theory and method of single channel seismic refraction are briefly outlined in Appendix 1.1.

4.2.2.2 Cut Area B.

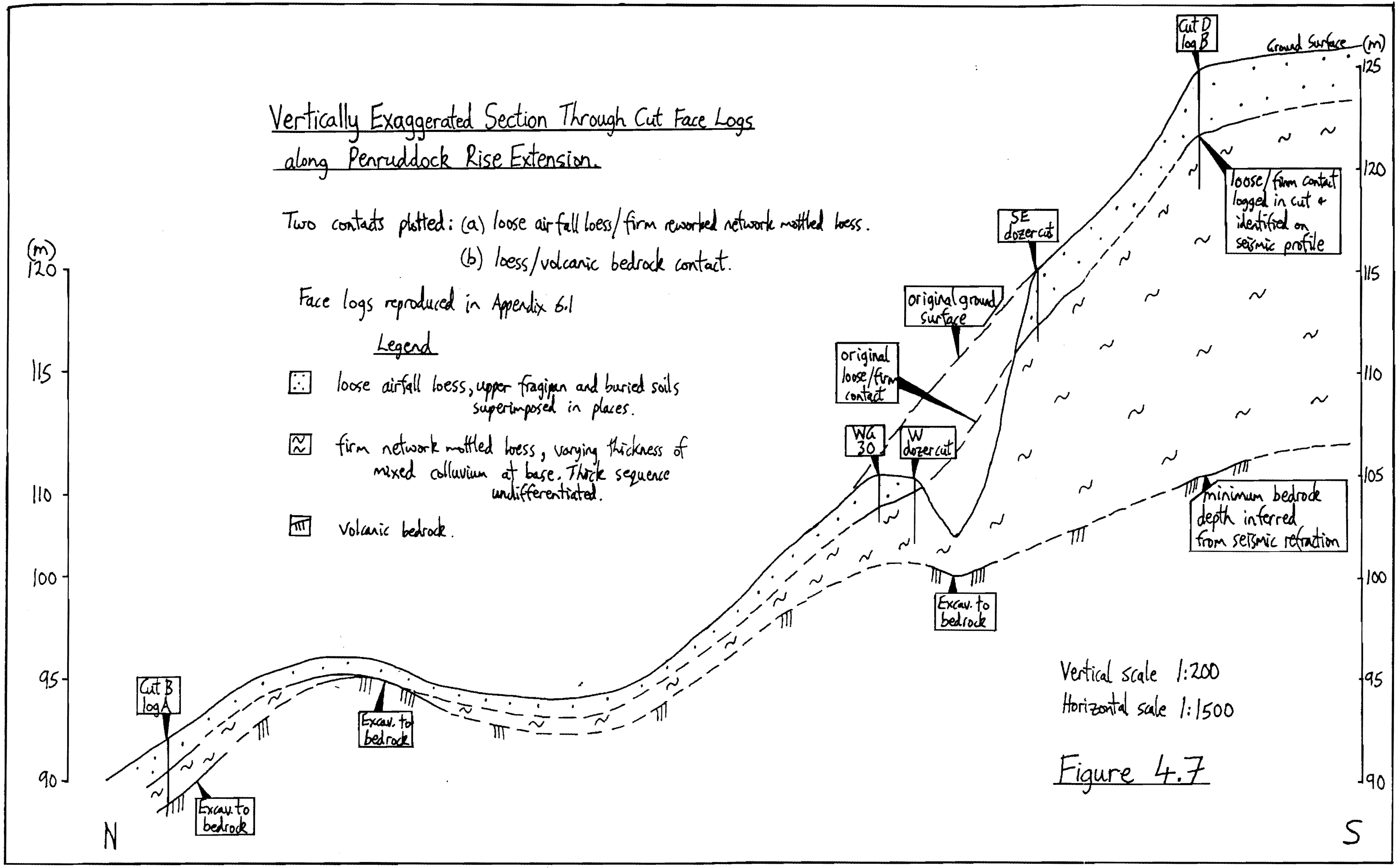
A radial depth profile for cut area B (Figure 4.4) shows a 2 layer interpretation, with loess (v=350m/s) underlain by basalt bedrock (v=2400m/s). The contact dips gently, parallel to the surface slope, but is irregular with loess thickness varying from 2-3m. This interpretation gives a depth to bedrock consistent with the geomorphic setting (i.e. a roll on the spur) and test pit data from nearby. The results indicate that the proposed cut depth is possible, but service trenches (sewer, stormwater) may require some excavation of bedrock.

4.2.2.3 Cut Area D.

Travel time data for the 50m line on cut area D was of poorer quality (more scatter of data) than the profile run on cut area B. A 2 layer case with surface layer velocity of 365m/s and refractor velocity of 700m/s was adopted. These velocities are within the range of velocities for loess material (Table 1.1), which are likely to be controlled by the two major physical variables in loess (i.e. dry density and moisture content).

The radial depth plot of the 700m/s refractor (Figure 4.4) shows a wavy (1-2m amplitude) surface that dips at a slightly shallower angle than the surface slope. This surface could be a transition from dry to moist loess, the top of a well developed fragipan, or the boundary between loose and compact loess (i.e. air fall and insitu reworked loess). The upper fragipan, where present in this area, is thin (<0.5m) and close to the surface (<1m), which does not conform with the refractor profile. A transition from dry to moist loess might be expected to be a less wavy and more gradational zone not offering sufficient velocity contrast to produce a refractor.

The boundary between loose and dense loess is the most likely explanation for the refractor. A gently dipping, sharp, wavy contact between loose and dense loess was logged in excavations across the south flank of the gully (Figure 4.6) and a similar contact was noted in excavations logged on cut area D (Figure 4.7 and Appendix 6.1, S dozer log & cut D log B). Depth to the loose/dense contact corresponds with



Vertically Exaggerated Section Through Cut Face Logs along Penruddock Rise Extension.
Two contacts plotted: (a) loose airfall loess/firm reworked network mottled loess.
(b) loess/volcanic bedrock contact.
Face logs reproduced in Appendix 6.1
Legend
loose airfall loess, upper fragipan and buried soils superimposed in places.
firm network mottled loess, varying thickness of mixed colluvium at base. Thick sequence undifferentiated.
Volcanic bedrock.
(m)
120
115
110
100
95
90
Cut B log A
Excav. to bedrock
Excav. to bedrock
WG 30
W dozer cut
original loose/firm contact
original ground surface
SE dozer cut
Excav. to bedrock
Cut D log B
Ground Surface
(m)
125
120
115
110
105
100
95
90
loose/firm contact logged in cut + identified on seismic profile
minimum bedrock depth inferred from seismic refraction
Vertical scale 1:200
Horizontal scale 1:1500
N
S


Figure 4.7

radial depth to the refractor at the SE end of the seismic line. Shallowing of the refractor in a down-slope direction is consistent with removal of overlying material by the large slope failure discussed in section 4.2.1.3.2.

Note that a bedrock refractor was not identified along the 50m line. Critical distance calculations, using a simple 3 layer model, give a minimum loess thickness of 18m. The maximum cut depth was 8m without intercepting bedrock, so the inferred loess thickness of 18m remains untested.

### 4.2.3 Summary.

(1) Three major engineering geological units were mapped along the road alignment and gully area: (a) volcanic bedrock - flows and dykes of basaltic lava, (b) insitu loess - airfall and locally reworked loess capped by a well developed upper fragipan, (c) loess colluvium - reworked insitu loess without fragipan and prone to severe gully and rill erosion.

(2) The large gully (crossed by fill area D) can be divided into 3 segments trending in different directions. The lower part is a pre-loess gully incised into bedrock , with actively eroding loess colluvium on the flanks. The middle part is eroded to a bedrock base level, with flanks of stable insitu loess. The active upper part is eroding headwards up the spur, with flanks developing by small slide/flow slope failures.

(3) Cut area D crosses the degraded head scarp of a large inactive slope failure. Loess thickness at this point is approximately 18m - cut depth 8m.

(4) Loess thickness over cut area B ranges from 1-3m, which is sufficient to allow the proposed cut, but service trenches may require excavation of limited amounts of bedrock.

(5) The gully at fill area D is stable with no evidence of recent or active erosion of the flanks.

## 4.3 GULLY FILL.

### 4.3.1 Introduction.

The major work of the author at Westmorland was the logging of fresh exposure in cut areas and monitoring fill operations in the gully (fill area D). Day to day supervision of the filling programme was carried out by the clerk of works for the project civil engineers (P.J. Kingsbury), while the author assisted, advised and carried out field and laboratory testing as required. The following is a brief description of events involved in constructing the loess fill.

A backhoe cleared topsoil and vegetation from the gully walls, cut a 3m wide channel in the floor of the gully, and a track down the flank to allow machinery access. The insitu loess was also ripped to give improved bonding with the fill material (Figure 4.8). The first 1/3 of the fill material came from cut area B, with the remaining 2/3 sourced from cut area D. Topsoil and dry loess was removed from cut areas by scrapers and bulldozer. Moist insitu loess (W=10-15%) was cut and transported by scrapers, spread 20-40cm deep by bulldozer and compacted by a sheepsfoot wheeled compactor (Figure 4.9). After placing 2-3m of fill a trench was dug along the gully center line to place a temporary drainage pipe. The temporary pipe was to accommodate any surface water flow in the gully during construction of the fill. During the final stages of fill construction an intake structure for the subdivision stormwater system was built in the gully upstream of the fill, and the temporary pipe was blocked up. The completed fill has a downstream batter slope of approximately $20^0$ and 15m high, and an upstream batter slope of approximately $30^0$ and 3m high. Maximum vertical thickness of fill is approximately 7m.

There was no provision for watering of fill, so supervision was aimed at ensuring only loess of adequate moisture (W=10-15% in cut) was used and that lift thickness/compactive effort was sufficient to give even compaction.

### 4.3.2 Density Testing of Fill.

#### 4.3.2.1 Importance of Quality Control.

Daily supervision of cut and fill operations, backed up by regular tests for dry density and moisture content of the fill (and occasionally the cut areas) is vital for good quality control. Inadequate design and control of cut and fill operations has been the cause of many stability problems in loess materials on Banks Peninsula. Documented examples include a residential site on Quarry Road, St Andrews Hill (D.H. Bell pers comm, 1988) and 3 road sites on the Port Hills and Diamond Harbour Road (Yetton, 1986).

The Quarry Road site involves differential settlement and partial collapse of a swimming pool and house built on loess fill dumped from

Figure 4.8 : View Down Gully Prior to Filling. Channel cut at base of gully is to allow machinary access for initial filling

Figure 4.9 : View Up Road Extension. Bulldozer spreading and sheepsfoot compactor rolling fresh loess fill sourced from cut area D in background. Dark colour of fresh loess indicates moisture content in the 10-14% range

rock quarry operations circa 1900. Tunnel gullies and settlement developed because of the variable nature of the uncompacted fill. Two major factors were identified as leading to the problems: (a) no attempt was made to compact the original fill, (b) investigations at the building stage failed to identify the fill and associated potential problems.

The road sites described by Yetton (1986) are Parklands Drive (Port Hills), Andersons Corner (Diamond Harbour Road) and Hendersons Culvert (Diamond Harbour Road). Stability problems at these sites were caused by collapse of deep seated tunnel gullies developed in loess fill placed as road embankments. Inadequate roadside water table and culvert design and maintenance, along with some what variable fill quality were seen as the major factors involved in tunnel gully development.

In summary, poorly compacted loess fill is prone to severe erosion and collapse, and adequate control of storm-water and natural drainage is important for preventing initiation of erosion in loess fill materials.

#### 4.3.2.2 Acceptable Standard of Fill.

A commonly used method of obtaining a standard for compaction of fill is to carry out standard compaction tests using a Proctor mould . Compaction tests over a range of moisture contents give a curve that defines a maximum dry density at an optimum moisture content. Proctor tests were not run on loess from the cut areas because of the consistency of past results from different parts of the Port Hills. Table 1.4 gives recompacted dry densities for a range of studies as 1800 to 1900 kg/m$^3$, optimum moisture content is 13% to 14%. The lack of variation in values can be attributed to a similar lack of variation in grainsize distribution and clay type (also reflected in Atterburg limit data). Given the above information it is reasonable to adopt maximum dry density of 1850 kg/m$^3$ and optimum moisture content of 13.5% for recompacted Port Hills loess.

The cost of maintaining optimum moisture content/ maximum dry density conditions on a small project, such as the Penruddock Rise extension, must be weighed against the purpose of the fill (e.g. earth dam vs. suburban road embankment) and its performance compared to undisturbed insitu loess. An approach was adopted where visual inspection and the good judgement of the earthworks supervisor and experienced machine operators is confirmed by occasional density testing.

Moist insitu loess cut at W=10 to 14% will compact at W=8 to 12% (moisture loss due to transport and spreading of lift) forming a massive evenly compacted fill, with dry density significantly higher than insitu material (1500-1700 kg/$m^3$ insitu vs. 1700-1900 kg/$m^3$ recompacted). It was considered that fill of the quality described above would perform at least as well as insitu loess. Maintining continuity of fill placement is important when utilising the natural moisture content of cut material. Dry material developed on cut and fill surfaces, because of delays or breaks in the filling programme, must be removed before recommencement of fill placement.

4.3.2.3 Comparison of Density Tests.

Daily supervision of fill placement was supplemented by direct testing of dry density and moisture content at approximately 1m intervals in the fill. A Troxler nuclear densometer (ND) was used on 6 occasions (5/2, 24/2, 29/2, 28/3, 3/5, 17/5/88) to take density and moisture content readings in the top 300mm of fill. Three to five sites were tested at each visit, with three 35mm diameter stainless steel sampling tubes collected from each site. Tube samples are used to determine density in insitu materials at Westmorland and Coleridge Tce (Chapter 3) and are a convenient physical test to use as a comparison to the nuclear method. Tube samples were also compared to the Balloon Densometer (BD) and Sand Replacement (SR) methods. BD and SR were not compared directly to ND because of the time involved in running all 3 tests together. The ND machine was hired by the hour and fill placement had to be suspended (because of the small size of the area) while testing was carried out, so time was an important economic factor. Tube, BD and SR tests were carried out on Sundays at the leisure of the author (see Appendix 1.2 for test methods and Appendix 5 for tables of results).

Comparisons of results have been made by simply plotting bulk densities, dry densities and moisture contents for each of the methods used side by side (see figures 4.10 and 4.11) Moisture contents for the physical tests (BD, SR, tube) show very little scatter from a 1:1 correlation, which might be expected as all methods use a wet weight-oven dry-dry weight procedure to determine moisture content (W%).

Density comparisons for the physical tests show up a difference in sampling. The BD and SR tests both require excavation of a 100mm diameter by 100mm deep hole in the fill. The 35mm diameter by 150mm long tube samples a smaller surface area and a potentially greater number of layers, thereby resolving minor local variations in density that are averaged out by the larger diameter, larger volume BD and SR samples. A BD vs. SR plot shows little scatter about a 1:1 correlation line, while plots of BD vs.

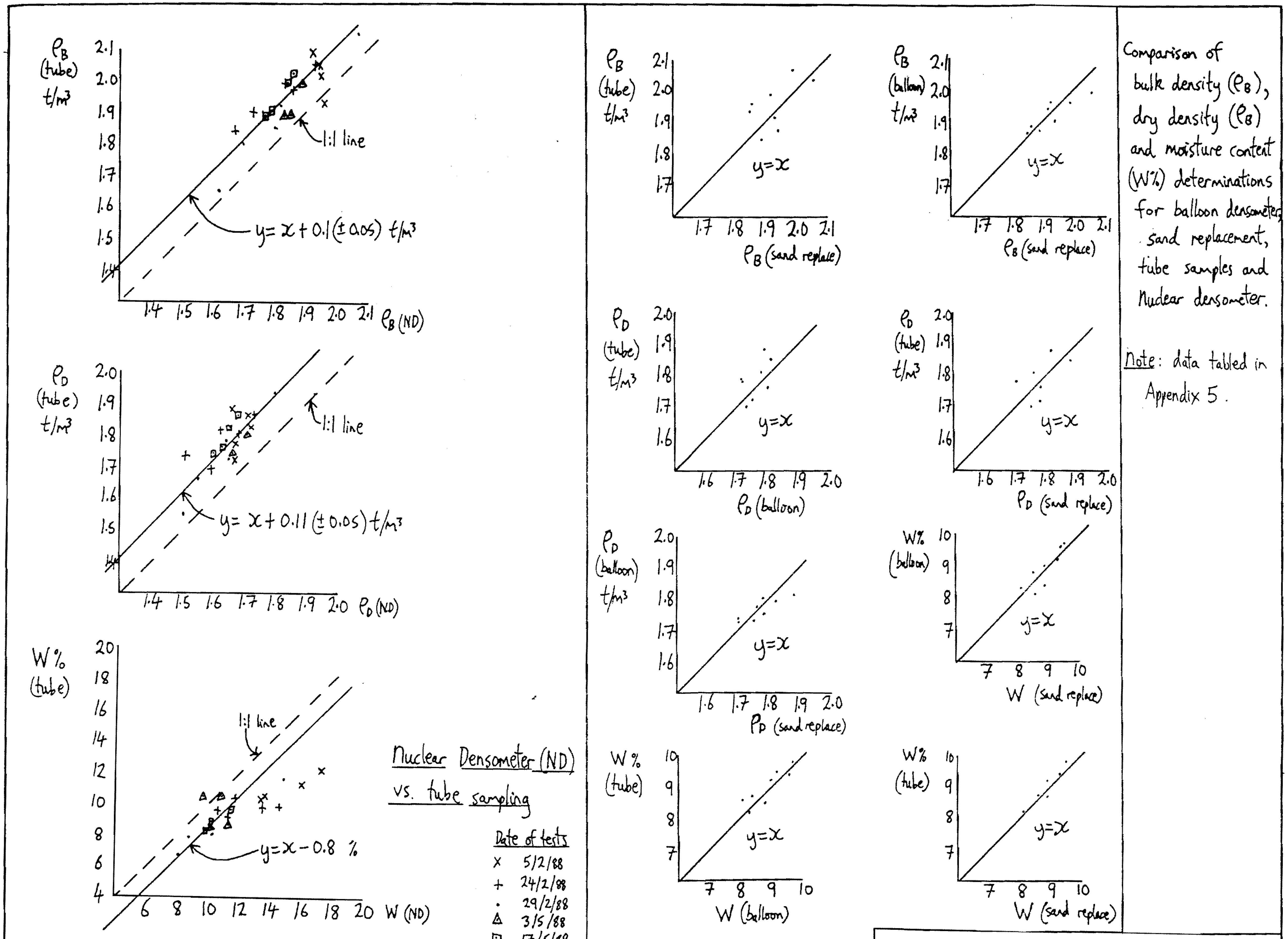



Figure 4.10 : Comparison of Density Test Methods

Figure 4.11 : Section of Trench Cut in Lower Half of Gully Fill. Shows moist massive fill overlain by blocky layered slightly moist fill. Partings between blocks are fracture discussed in fill stability and triaxial testing

| Sample | Dry Density ($kg/m^3$) | insitu W% | Range $\sigma_3$ (kPa) | c (kPa) | ø (°) |
|---|---|---|---|---|---|
| Cut 1 | 1720 | 13.5 | 50 – 120 | 44 | 31 |
| Fill 1 | 1800 | 8.5 | 50 – 175 | 178 | 30 |
| Fill 2 | 1650 | 8.0 | 50 – 150 | 32 | 29 |

Table 4.1 : Triaxial Test Results (P-Q plots in Appendix 4.2)

tube and SR vs. tube show a broader scatter about the line. Given the small number of samples (11) the 3 physical tests appear directly comparable.

Having established the tube sample method as representative of physical density/moisture content tests, it can be compared to ND tests carried out on the fill. A line of best fit can be placed through the data points on the tube vs. ND dry density plot. The relationship is : tube dry density = ND dry density + 120 ( 50) $kg/m^3$. A similar line can be fitted to the tube vs. ND bulk density plot, with a relationship : tube bulk density = ND bulk density + 100 ( 50) $kg/m^3$. These relationships compare favourably with the tube density vs. BD and SR density where the relationship is : tube density = BD or SR density ( 50) $kg/m^3$. For Westmorland loess physically measured density = nuclear measured density + 100 $kg/m^3$.

The relationship of moisture content for tube and ND is not immediately clear. Some suites of samples (i.e. samples collected on the one day) show a relatively constant ND moisture content and wide ranging tube moisture content, while other suites show constant tube moisture content and wide ranging ND moisture content. Excluding 5/2/88 data from the plot is favoured, because high ND moisture contents (13-18%) are not realistic. These moisture contents are approaching the plastic limit and past experience with laboratory and field samples suggests a moisture content of 10-12% (estimate in field note book) for the material in question. The Nd is being influenced by a factor (or factors) other than the soil water content.

Ruygrok (1988) suggests that relatively high amounts of organic material will give high readings of moisture content, because of the organic hydrogen adding to the count of water hydrogen. It is possible that ND tests giving anomalously high moisture contents could have been run in dominantly brown grey silty loess, i.e. buried soil type loess. This material has been found to have relatively high amounts of fine disseminated organic fragments during pretreatment for grainsize analysis (see section 3.2.4).

For Westmorland loess : physically measured moisture content = nuclear measured moisture content - 0.8% (+ an unknown correction factor for high organic content). Variations in cut material (e.g. fragipan, airfall loess, brown grey buried soil) preserved in the recompacted fill may cause anomalies in moisture content determined by ND.

### 4.3.3 Triaxial Testing.

#### 4.3.3.1 Introduction.

Triaxial strength tests were run on 3 suites of samples from the cut and fill operations at Westmorland. Material was sampled by 35mm diameter, 150mm long stainless steel drive tubes and tested under quick (failed at 1mm/min) UU (unconsolidated undrained) conditions. See Appendix 3.1 for an outline of method and Appendix 3.2 for discussion of alternative strength tests.

The original aim was to compare strengths of insitu cut material and recompacted fill material, thereby testing the assumption that loess recompacted at higher densities than insitu would perform at least as well as the insitu material. A secondary aim was to gain experience in sample collection, handling and testing for the major strength testing programme planned at Coleridge Tce (Chapter 3). The third suite of samples was collected to sample partings parallel to the fill surface, i.e. dipping 5-10$^0$ down-gully, and compare strength of partings with massive recompacted fill.

The partings mentioned above were noted in a layer of fill exposed in the temporary drainage pipe trench at the fill base. A 0.5 to 0.8m thick layer of fill had been compacted with a moisture content of approximately 6-8%, giving some of the lowest recorded dry densities for fill (1550 to 1700 kg/m$^3$, see Appendix 5 29/2/88). While dry density values are comparable with values for insitu loess the material is too dry to compact into a massive fill. The material forms crumbly clods, and sheeps- foot roller indents are preserved between individual lifts as open partings (see figure 4.12). A first step in considering the significance of a layer of partings on the overall stability of the fill was to compare parting strength with fill material strength.

#### 4.3.3.2 Results and Implications for Fill Stability.

Results of the 3 suites of triaxial tests are given in table 4.1 Cut1 samples were tested to compare insitu materials with recompacted fill (Fill1), but direct comparisons cannot be made because of different moisture contents (Cut1 13.5%, Fill1 8.5%). However Cut1 and Fill1 cohesion and friction angle values are plotted against moisture content and fit the trends of Coleridge Tce samples (see figure 3.7), indicating that the materials have similar strengths given similar moisture contents.

Fill2 samples were collected from the fill surface, with tubes angled at approximately $40^0$ to the slope, so partings were orientated $45\text{-}50^0$ to the tube sample axis and would fail when loaded in the triaxial apparatus. Fifteen samples were taken, and nine extruded samples displayed partings suitable for testing. The samples were failed over a range of confining pressures of 50-220 kPa. Four samples failed along partings at confining pressures of 50-150 kPa, giving the cohesion and friction angle results in table 4.1. Two samples failed partly on partings and partly through material at confining pressures of 170 and 180 kPa, while three samples failed through intact material at confining pressures above 200 kPa.

The details of testing outlined above suggest that partings are only significant at confining pressures below 150 kPa. Above 150 kPa partings are compressed to the stage that they no longer represent a relative plane of weakness in the fill material.

Cohesion and friction angle results for the 50-150 kPa range (i.e. 32 kPa, $29^0$) show that the frictional characteristics of the loess material are preserved, but cohesion is reduced (though still significant at 32 kPa) on partings. When the size of the triaxial sample (parting length 50-60mm) is compared to the discontinuous anastomosing nature, and the shallow ($5\text{-}10^0$) dip of partings in outcrop (see figure 4.12), the difference in strength between massive fill and parting layered fill becomes insignificant. Furthermore, the suggestion that partings are no longer a plane of weakness at confining pressures above 200 kPa (W=8%) leads to the possibility of similar behaviour at lower confining pressures (50-150 kPa) and higher moisture contents (W=10-14%).

The dry parting layer of 0.5-0.8m thick is covered by 5-6m of fill in the completed embankment, i.e. overburden pressures of 100-120 kPa apply. With equilibration of moisture contents to 10-14% over a 1-2 year period (similar moisture to insitu loess at depth) the dry parting layer may become consolidated, moist, massive loess similar to the under- and overlying material, i.e. the partings will no longer exist as planes of weakness that could threaten the stability of the gully fill.

#### 4.3.3.3 Summary.

(1) Insitu loess from the cut area and recompacted fill material were tested at different moisture contents, but results follow the trend of Coleridge Tce results (figure 3.7) and indicate similar strengths given similar moisture contents.

(2) Parting strength on tube sample scale (fill2 samples) is lower than

material strength because of reduced cohesion. Observations from fill2 testing suggest partings cease to be a relative plane of weakness at higher confining pressures.

(3) Discontinuity and low dip of partings on outcrop scale reduce strength differences between massive fill and parting layered fill to insignificant amounts.

(4) Overburden pressure and increased moisture content in the completed embankment may alter the parting layered fill to produce essentially massive fill material.

(5) Triaxial testing (combined with density testing and field observations) indicates that the loess fill has similar, if not improved strength properties to dense (dry density 1600-1800 kg/$m^3$) insitu loess.

### 4.3.4 Fill Stability.

Density and strength tests carried out on the gully fill (fill area D) confirm that it will perform at least as well as the insitu loess that it is sourced from. Engineering geological mapping of the area (section 4.2.1) shows there are no foreseeable geologic or geomorphic factors that will adversely effect fill stability.

Potential instability of the fill could occur if high moisture contents (i.e. approaching the liquid limit-20%) became established in the bank in discrete layers (or over wide zones), or if tunnel gullying and rill erosion caused undermining of the embankment. Control of stormwater run-off in the upper part of the gully, and from the road running across the fill is vital to prevent either erosion or the establishment of high soil moisture contents. Sound engineering design and careful construction of stormwater control, intake and disposal systems will ensure long term stability of the gully fill.

## 4.4 CUT FACE LOGS AND INDEX TESTS FOR INSITU LOESS.

### 4.4.1 Introduction.

Freshly cut faces were logged on cut area B, cut area D and fill area D (gully fill). Samples were collected from the gully area (WG-Westmorland Gully samples) and cut area D (CD-Cut area D samples). Positions of sample sites and cut face logs are shown on figure 4.4. Index tests were similar to those used at Coleridge Tce. (section 3.3.2), and methods are outlined in Appendix 2. Material index values and loess layering interpretations are also similar to those for the Coleridge Tce loess bank, so the following sections are often cross referenced to Chapter 3.

### 4.4.2 Loess Layers.

The following loess layer types were recognised in freshly cut faces of material mapped as insitu loess. Composite columns from logged faces are reproduced in Appendix 6.1.

(1) Network Mottled Loess is locally reworked yellow grey to brown grey silt. A description and inferred mode of formation is as given for Coleridge Tce loess (section 3.2.4), with thick sequences (2-3m) of network mottled loess show complexly interfingered layers and lenses, togeather with different ratios of yellow grey to brown grey silt and either sharp or gradational contacts. This complexity records local variations in erosion and redeposition during the period of loess accumulation.

(2) Buried Soil Type Loess , or brown grey silts are old topsoils relatively high (compared to other insitu loess) in finely disseminated organic material and worm burrows ( especially at the base of a unit). Observed buried soils vary in thickness from 100-300mm. Figure 4.5 shows a stack of buried soils, with splitting and reamalgamation of units. Further evidence for interpreting these layers as buried soils is given in section 3.2.4.

(3) Fragipan horizons at Westmorland have similar properties to fragipans described from Coleridge Tce (section 3.4.2). An upper fragipan caps most of the area mapped as in situ loess, while a buried fragipan was identified at the base of cut faces in cut area D (see figure 3.4). Fragipans at Westmorland differ from those at Coleridge Tce by having a thinner (<0.5m), coarser orange mottled lower zone and lacking an upper polygonally cracked zone.

(4) Airfall Loess is massive, loose yellow grey silt with relatively low clay content. Internal structure is limited to occasional worm burrows and rare to common calcareous rootlet casts. The low density and massive structure suggest this is true insitu loess, preserved as-deposited from an original dust cloud.

Airfall loess was logged, in many of the cut faces along the road alignment, overlying firm (or dense) network mottled and brown grey silt type loess (see figure 4.6). The contact between loose and firm is sharp, gently dipping and wavy (amplitude 0.5-1.0m). A vertically exaggerated section, through several cut logs, traces out this contact, but without attempting to show the wavy nature (see figure 4.7). This contact is interpreted as an erosional surface buried at the onset of the final period of loess deposition (recorded as the thick upper airfall loess). The occurrence of Moa bone fragments (exposed in scraper cuts) at this horizon also supports the interpretation given above. Note that upper fragipan and soil development have altered the top 0.5-1.5m of the airfall loess layer.

(5) Alluvial Loess is relatively dense, very clean (low clay %) , and relatively coarse. This material is recognised in small channel fills and wedges, or cones. Figure 4.6 shows an alluvial wedge within brown grey and network mottled loess.

### 4.4.3 Index Test Results and Trends.

Results of index tests are very similar to those obtained for Coleridge Tce samples, except that Westmorland loess is generally less dense and has a coarser grainsize. The range of dry density for Westmorland loess is 1400-1800 kg/m$^3$, compared to 1500-1900 kg/m$^3$ for Coleridge Tce loess. Grainsize of the silt/sand fraction, i.e. original airfall component of the loess, is coarser for Westmorland. This can be simply illustrated by comparing sand fractions for Westmorland and Coleridge Tce (Westmorland sand 11-17%, Coleridge Tce sand 11-12%). Plots of grainsize distribution curves are reproduced in Appendix 7.2.

Trends in index values can be defined by grouping the samples according to interpretation of loess layer type. This has been done in table 4.2. There is some overlap of index values between loess types, but generally one or more physical properties combined with careful observation and description is enough to define a loess layer type. Detailed results for index test values are given in Appendix 6.2.

### 4.4.4 Conclusion.

Loess deposits at Westmorland can be divided into insitu loess and loess colluvium. Insitu loess may be subdivided into 5 loess layer types that are identified by detailed logging of the fresh cut faces. Loess layer types are formed by the processes of deposition from airfall, colluvial and alluvial reworking, soil formation and weathering/ fragipan formation. Broad variations of index values for loess are explained by subdivision into loess layer types.

| Loess layer | Sample numbers | Insitu W% | Dry Density (kg/m³) | Plasticity Index (%) | Grainsize (%) | Crumb (class) | Pinhole erosion |
|---|---|---|---|---|---|---|---|
| Weathered above-fragipan loess | CD1, WG7, WG27. | 4 to 6 | 1580 to 1640 | 3 to 7 | clay: 15-17<br>sand: 11-13 | 1,2 | E180 to E360 |
| Mottled fragipan | CD8, CD2, Cut1. | 9.5 to 14.5 | 1720 to 1850 | 7 to 9 | clay: 17-26<br>sand: 11 | 1 | E >1000 |
| Airfall loess | CD3, CD6, WG21, WG23. | 7.5 to 13 | 1410 to 1580 | Non-Plastic | clay: 8-14<br>sand: 14-17 | 2 | E50 to E180 |
| Reworked insitu loess | CD4, CD7, WG10, WG30, WG31, WG32 | 7 to 15 | 1600 to 1720 | 2 to 5 | clay: 13-17<br>sand: 12-19 | 2,3 | E180 to E360 |
| Loess colluvium (in gully floor) | WG4, WG25. | 19 to 22 | 1650 | 5 | clay: 13-15<br>sand: 14-21 | 1 | E360 |

Table 4.2 : Index Test Results Grouped by Loess Layers

4.5 SUMMARY.

(1) The Penruddock Rise extension of Westmorland subdivision runs up a gently to moderately dipping, loess draped basalt spur on the West flank of the Port Hills. Two areas of cut and two areas of fill maintain an even grade on the road. A large gully, that dissects the SW flank of the spur, was filled with upto 7m of loess material from the cut areas.

(2) Erosion (section 4.2.1) in the lower gully is limited to tunnels and open gullies in loess colluvium on the bedrock controlled side walls. The upper gully is actively eroding headwards, with side walls of insitu loess being modified by small ($<20m^3$) slide/flow failures. The middle section, where the loess fill was placed, appears to be stable (in terms of active downcutting and side wall development).

(3) Single channel seismic refraction was successful at determining basalt bedrock profiles where loess is upto 10m thick (section 4.2.2.2), but unsuccessful for loess 10-20m thick (section 4.2.2.3). Marked density changes (airfall loess, 1400 $kg/m^3$ to reworked/buried soil loess, 1700 $kg/m^3$) can be identified in thick loess sequences (section 4.2.2.3).

(4) Comparisons of four density/moisture measuring techniques were carried out during monitoring of the gully fill operations (section 4.3.2). Physical tests (tube sampling, Balloon Densometer, Sand Replacement) are directly comparable.

Nuclear testing (Nuclear Densometer) and physical tests have the following relationships : (a) physical density = nuclear density + 100 $kg/m^3$, (b) physical moisture content = nuclear moisture content - 0.8 % (+ an unknown correction factor for relative organic content). The organic content is included in the loess, rather than being contamination by modern top soil, or plant remains.

(5) Triaxial testing (UU method) was used to compare the relative strengths of insitu loess/recompacted fill, and massive fill/parting layered fill. Results show no significant difference in strength between insitu loess and massive fill. Parting layered fill had reduced cohesion compared to massive fill, but observations from triaxial testing and trench exposure suggest that the partings will have no affect on the overall stability of the fill embankment.

(6) The gully fill is of sufficient quality to perform at least as well as insitu loess. Stability of gully fill (section 4.3.4) is dependent on control of stormwater in the upper gully, and on the road surfaces across the cut and fill area. Surface water flows (including house roofs, sealed areas) must be collected and disposed of in the subdivision stormwater system.

(7) Two additional loess layers were identified over the three discussed in Chapter 3 (Coleridge Tce).

Airfall loess is massive, loose, low clay loess, with the lowest dry density encountered in this study (1400-1500 kg/m$^3$). This material is inferred to have an original airfall structure, with minimal colluvial reworking or soil/fragipan type alteration.

Alluvial loess is a minor component of reworked insitu loess that is recognised in channel fills and alluvial cones (or small fans).

(8) Index test results are similar to those obtained from Coleridge Tce loess. Westmorland loess is generally coarser and less dense than Coleridge Tce loess. Dry density ranges are 1400-1800 kg/m$^3$ (Westmorland) and 1500-1900 kg/m$^3$ (Coleridge Tce), while the sand fraction of Westmorland loess is upto 7% higher than Coleridge Tce loess.

# CHAPTER 5 : SUMMARY AND CONCLUSIONS.

## 5.1 McCORMACKS BAY QUARRY SUBDIVISION.

(1) The residential sites included benches and batter slopes of a disused volcanic rock quarry, which were unmodified since cessation of quarrying.

(2) Potentially unstable blast-shattered bedrock and loose colluvium on the batter slopes was the main risk to residential development. Field evidence suggests that blast damage was caused by excessive use of explosives in 'bulled holes'.

(3) Remedial measures involved hand-clearing of large loose blocks and trimming of slopes with a backhoe. Reinforced concrete buttressing, non-pressurised grouting and a gabion basket retaining wall were used to support potentially unstable material that could not be removed because of the risk of undermining structures on higher benches. A design-as-you-go approach was adopted for remedial measures because of the unpredictable sub-surface distribution of volcanic bedrock units.

(4) The complexity and cost of remedial measures could have been reduced if they were carried out in a single project before lot boundaries were fixed and lots were sold to individuals.

## 5.2 COLERIDGE TCE LOESS BANK.

(1) The bank at Coleridge Tce is a cross-section through the top 8m of a broad loess spur. Loess thickness is inferred as 10-20m.

(2) Chimney erosion, open gully erosion, slab failures and small ($<20m^3$) slide/flow slope failures have eroded the bank a maximum of 3m from its original cut position (cut approximately 40 years ago). A sealed road with stormwater drainage, as well as thick vegetation above the cut bank has reduced the amount of erosion compared to unprotected banks.

(3) Erosional processes, loess layering in the bank and physical tests of materials (see sections 5.6, 5.8, 5.9) were combined to produce an engineering geological model to aid in design of an anchored retaining wall. Bank drainage and stormwater control are crucial to the long term stability of the bank, and maintenance of low soil moisture will mean the retaining role of a wall is minimal.

## 5.3 WESTMORLAND SUBDIVISION EXTENSION.

(1) Two areas of cut and two areas of fill occur along the Penruddock Rise extension on a loess draped basalt spur. 7m of loess fill were placed in a large gully that dissects the SW flank of the spur.

(2) The gully and cut areas are in relatively stable insitu loess, and the gully fill is of sufficient quality to perform at least as well as insitu loess. However, long term stability of the gully fill is dependent on careful and thorough control of stormwater.

## 5.4 ENGINEERING GEOLOGICAL MAPPING.

(1) Mapping onto 1:200 scale plans based on limited site surveys at McCormacks Bay was sufficient to record details of the volcanic bedrock units, and to delineate areas that would require remedial work. Sub-surface investigations on batter slopes would not have provided any useful information, considering the potential time and cost involved.

(2) Surveying and detailed (1:50 scale) logging of the bank at Coleridge Tce was successful at recording erosional processes and the detailed loess layering in the bank (section 5.9).

(3) 1:500 topographic plans were used to map volcanic bedrock, insitu loess and loess colluvium, as well as erosional features within these units. History of development of the gully, and active and relatively stable areas could be recognised from this mapping and an examination of low level air photos.

## 5.5 SINGLE CHANNEL SEISMIC REFRACTION.

(1) Refraction profiles were used at McCormacks Bay to determine the depth of fill on quarry benches. Results appeared acceptable (no depth control) and study of the data confirmed field observations about the variability in grainsize and sorting of the fill.

(2) Profiles run at Westmorland were successful at delineating a bedrock profile for loess cover <10m thick. A profile run across thick loess detected a contact between loose airfall loess and relatively denser reworked loess. The loess thickness was inferred to be at least 20m.

## 5.6 SHEAR STRENGTH OF LOESS.

(1) 35mm diameter tube samples were collected from Coleridge Tce and Westmorland for triaxial testing by an UU method. Samples were tested at confining pressures approximating expected overburden pressures, at insitu moisture contents (8-14%), and at raised moisture contents (upto 19%) that could become established at the sites.

(2) Results showed the internal angle of friction ($\phi = 30^0$) to be independent of moisture content changes, while cohesion ranged from c = 180 to 0 kPa for W = 8 to 19%. Cohesion decreased rapidly from W = 8% to W = 13%, with a more gradual decline to c = 0 kPa at W = 19%. This change in behaviour is probably related to the affects of soil suction and clay bridge bonding at different moisture contents.

(3) Comparative tests between insitu loess and loess fill at Westmorland show no significant difference in strength.

(4) Shear strength of loess is primarily controlled by insitu moisture content, with a possible minor contribution from material density and clay content.

## 5.7 DENSITY TESTING OF LOESS FILL.

(1) Physical density tests (tube samples, Balloon Densometer, Sand Replacement) were compared with a Nuclear Densometer during monitoring of the gully fill operation at Westmorland.

(2) Physical tests were found to have a y = x relationship (i.e. 1:1 correlation) when plotted against each other, but the scatter of data was greater with tube samples because they sample a smaller surface area and greater depth than the other methods.

(3) Nuclear and physical tests have the following relationships : (a) physical moisture content = nuclear moisture content - 0.8% , and (b) physical density = nuclear density + 100 kg/m$^3$.

## 5.8 INDEX TESTING OF LOESS.

(1) Index tests that had been commonly used in past studies, were carried out at Coleridge Tce and Westmorland. Crumb tests and Atterberg limits showed no variation or grouping consistent with the identified loess layers (section 5.9). Grainsize results could be put into broad groups, but with considerable overlap for some loess layers. Dry density and pinhole erosion results showed consistent grouping of values for

different loess layers.

(2) Comparisons of Coleridge Tce and Westmorland results shows that Westmorland loess is generally coarser and less dense than Coleridge Tce loess. This is possibly due to Coleridge Tce being on the lee slope of the Port Hills with respect to the dominant westerly winds.

## 5.9 INSITU LOESS LAYERING.

(1) Loess layering was recognised by careful and detailed logging of fresh faces of insitu loess, and confirmed by index test results. 5 loess layers have been identified as : (a) airfall loess, (b) buried soil type loess, (c) network mottled reworked loess, (d) alluvial loess, (e) fragipan.

(2) The important loess layers, in terms of engineering geology, are the fragipan, airfall loess and reworked insitu loess (b,c,d above) which exhibit significant differences in erodibility and permeability that may control slope processes in a loess deposit.

## 5.10 RECOMMENDATIONS FOR FURTHER WORK.

(1) Loess layers are defined from 2 restricted sites. The system should be further tested and modified by applying it to broader loess investigations. Understanding loess deposits in terms of depositional, erosional and soil/fragipan forming periods (along with collection of dateable material) will advance the general knowledge of Quaternary geology in New Zealand.

(2) 6 recommendations for further work to test the triaxial results and conclusions are given in section 3.4.4.

(3) Monitoring of soil moisture conditions, and fluctuations in these conditions in stable and unstable loess slopes will complement an understanding of shear strength parameters, and allow for accurate assessments of loess slope stability, or potential instability.

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# APPENDIX A
# PREVIOUS WORK

# APPENDIX A : PREVIOUS WORK.

## A.1 Engineering Geological Approach to Urban/Residential Development in the South Island.

Engineering geological input into residential development has increased in the last ten years, primarily because of greater awareness of potential problems brought about by disasters such as the Abbotsford Landslip and smaller scale problems on high density, high value subdivisions being developed on relatively steep and "marginal" land (Bell, 1984).

Several papers by Bell, and Bell and Pettinga (Bell,1984; Bell & Pettinga,1985; Bell,1987) develop an engineering geological approach from their experiences with subdivision and site investigations in the Queenstown area and on the Port Hills, Christchurch. Bell (1984) states : "Engineering Geological data input is essential to sound geotechnical practices, and the methods relevant to urban planning and development include site-specific mapping and air-photo interpretation, subsurface investigation where appropriate, and limited geomechanics testing of rock or soil materials."

Bell & Pettinga (1985) have a preference for national legislation, within the existing planning framework, to require engineering geological and/or geotechnical input at concept and scheme plan stage. This input is important for identifying geological limitations to the final layout of the subdivision, and the identification/investigation of potential hazards. Often the Engineering Geologist is called in at the end of stage D, or at stages E&F (see Table 1.1) when development is already underway and options for avoidance or remedial measures to combat geological problems may be limited and expensive.

## A.2 Volcanic Bedrock Geology

The geology of Banks Peninsula was first described by von Haast (1879) who had access to the recently completed Lyttelton rail tunnel that exposed the layered nature of the Lyttelton volcano. Speight published numerous papers, dealing with petrographic and stratigraphic aspects of Banks Peninsula, between 1893 and 1944 (see Crampton, 1985). Sheet 21 of the 1:250000 geological map series, which includes Banks Peninsula, was published by Oborn and Suggate in 1959. Liggett and Gregg (1965) studied the stratigraphy of Lyttelton and Akaroa volcanos. Radiometric (K/Ar) dates for Banks Peninsula rocks were produced by Stipp and McDougal (1968).

In the last ten years most work on Banks Peninsula has been carried out by staff and graduate students of the Geology Department, University of Canterbury, Christchurch. Thiele (1983) mapped the basement geology of Lyttelton volcano in the Gebbies pass area, and Sewell (1985) studied the stratigraphy and chronology of central Banks Peninsula, particularly the relationship between the Lyttelton and Akaroa volcanos.

Summaries of the geological history of Banks Peninsula are provided in field trip guides by Weaver et al.(1985) and Weaver & Sewell (1986). These field trip guides, along with the recently published paper by Sewell (1988), provide the most recent interpretations of the volcanic history of Banks Peninsula.

### A.3 Bedrock Engineering Geology.

Engineering geological classification of Banks Peninsula volcanics is limited to the work of Crampton (1985) on Lyttelton volcanics encountered in the tunnel section of the Lyttelton to Woolston LPG pipeline (also see Bell and Crampton, 1986).

Waters (1988) mapped the Halswell quarry (Diamond Harbour Volcanics) and carried out geotechnical tests for aggregate use. Banks Peninsula volcanic rocks have been discussed in terms of hydrogeological properties by Sanders (1986) and Namjou (1988).

### A.4 Loess Deposits.

Von Haast (1879) was the first to recognise the yellow silts of Banks Peninsula as wind-deposited loess material of Quaternary age. Since von Haast there has been some controversy over the nature and origin of loess deposits in the South Island, and the matter is reviewed by Raeside (1964). Raeside's paper gives a broad view of the nature and stratigraphy of loess in the South Island.

Ives (1973) follows up Raeside's work looking specifically at Canterbury loess, while Griffiths (1973) deals with the stratigraphy and distribution of loess on Banks Peninsula. Tonkin et al (1974) discuss the significance of paloesols in Timaru loess. Goh et al (1977) look at improved methods for dating Banks Peninsula loess, and Goh et al (1978) apply similar methods to Timaru loess. Kohn (1979) identified a tephra found in a loess layer at Amberley (Nth Canterbury).

## A.5 Engineering Geology of Surficial Deposits.

Surficial deposits include loess, volcanic colluvium and modifications and mixtures of both. Reports on geotechnical properties of loess include Birrell & Packard (1953), Evans (1978), Bell & Evans (1981), and Bell (1983). Studies on slope processes in loess have mostly been biased towards tunnel gully development and are well documented by Yetton (1986) and Bell et al (1986).

Recent work on Banks Peninsula surficial deposits include the MSc Eng Geol theses of Scott (1979), Crampton (1985), Yetton (1986), Glassey (1986), Mackwell (1986), and Tehrani (1988). Published material includes Bell (1981), Bell et al.(1986) and Bell & Trangmar (1987), which deal with geotechnical properties, active processes and remedial measures in loess deposits of the Port Hills.

# APPENDIX 1

## FIELD METHODS

1.1 Seismic refraction
1.2 Fill Density Tests
1.3 rock and Soil Description

APPENDIX 1.1 SEISMIC REFRACTION METHOD

The single channel signal enhancement seismograph has a single fixed receiving geophone, with a movable hammer and plate energy source. An inertia switch on the hammer is used to start the timing circuit. The incoming signal is displayed on an oscillograph, allowing first arrival identification by eye. Signal to noise ratios can be increased (enhanced) by signal addition from repeated hammer blows. A sequence of hammer blows is depicted diagramatically in Figure A1.1.1.

The interpretation method used is a simple straight forward procedure called the reciprocal (Hawkins) method. It was first described for shallow refraction investigations by Hawkins (1961). Forward and reverse traverses are necessary for each line so the time-depth ($t_d$) principle can be applied. The forward and reverse times to a shot point (station) are added, the total travel-time for the line is subtracted and the total divided by two to give the travel-time from the surface station to the refractor beneath, i.e. the time depth (Figure A1.1.2)

The main advantages of the reciprocal method are the ease of calculation and the fact that the time-depth principle automatically compensates for irregularities in the ground surface and refractor. Step by step notes on the data reduction are given on the ENCI 472 hand out produced by Dr J.R. Pettinga in 1982 (Figure A1.1.3).

The major source of error encountered in the method is in the subtleties of determining velocities (i.e. reading the travel-time plots) and deciding weather certain features of the plots are significant or not. An accurate geological model is most important to correct interpretation of the travel-time data.

Figure A1.1.1 Theory of Operation for Single Channel Seismograph

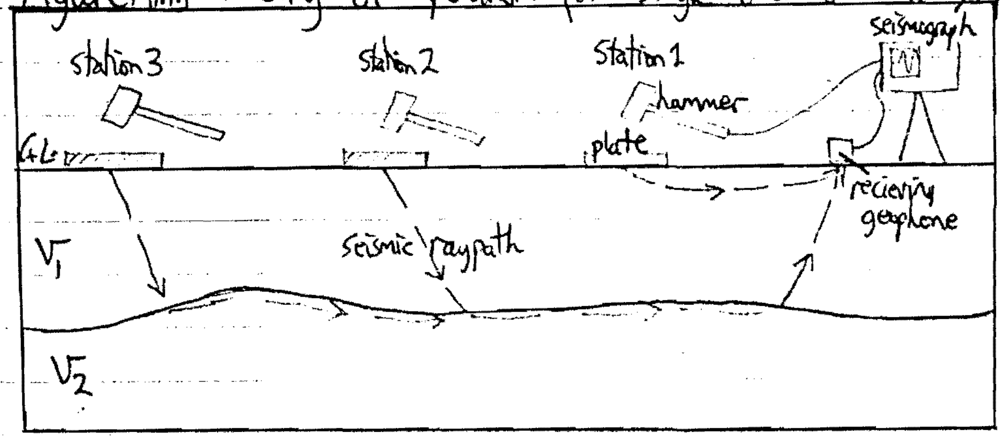
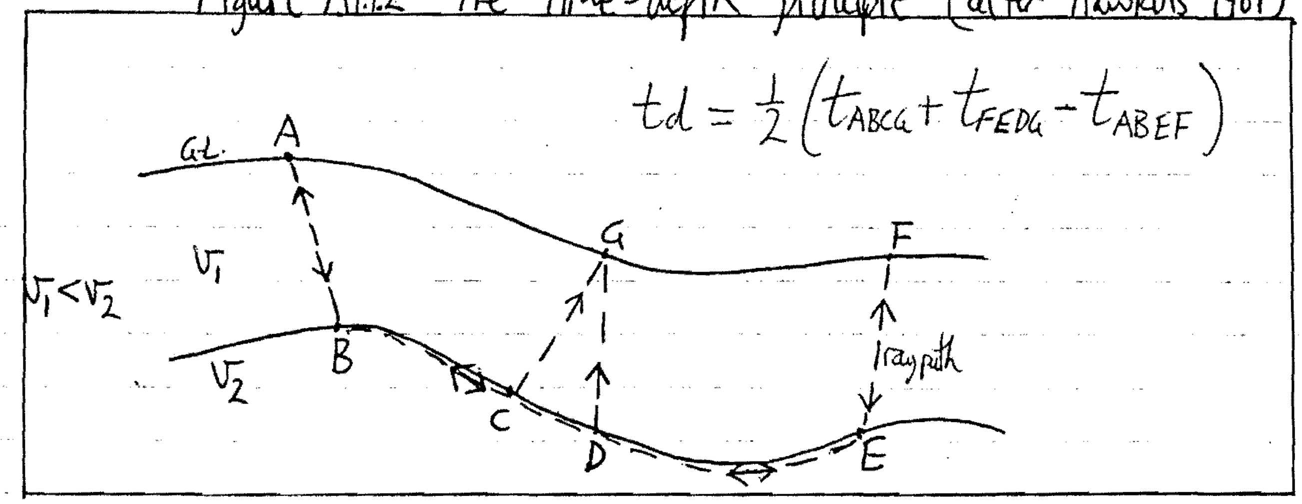



Figure A1.1.2 The time-depth principle (after Hawkins 1961)



ENCI 472

# THE RECIPROCAL METHOD FOR SHALLOW SEISMIC REFRACTION SURVEYS

## DATA REDUCTION PROCEDURE

**Step One**

$$T_R = \frac{t_{f\ total} + t_{r\ total}}{2}$$

$$t_d = \frac{t_f + t_r - T_R}{2}$$

where

$t_d$ = time-depth

$t_f$ = forward time

$t_r$ = reverse time

$T_R$ = reciprocal time

$t_f - t_d$ = t corr. f

$t_r - t_d$ = t corr. r

The recorded travel times of each spread are plotted as time-distance curves, and the lines of best fit drawn for the apparent velocities. A comparison of the time-distance curves for the forward and reverse directions and a direct comparison for the two shot-point to spread distances for each direction are made. An averaged reciprocal time is obtained from the extended travel-time profiles. The time depths to the important refractor are then computed on a table and the recorded travel times are corrected by the subtraction of the corresponding time-depths. The corrected travel times are to a point on the refractor beneath each station.

**Step Two**

Plot the forward and reverse corrected travel times on a separate sheet and identify the various matched segments of the graphs that approximate straight lines.

**Step Three**

Calculate the seismic velocities from the slopes of the lines for each segment representing the refractor, and establish the upper layer seismic velocity from the uncorrected plot.

**Step Four**

Calculate the depth conversion factor (D.C.F.) from the expression

$$\text{D.C.F.} = \frac{V_2}{\sqrt{\left(\frac{V_2}{V_1}\right)^2 - 1}}$$

for each 'bedrock' type represented along the refractor identified from graph segments.

**Step Five**

Multiply the depth conversion factors by the time-depths to determine the radial depth to the seismic interface.

**Step Six**

Plot the section-line profile of the ground surface and seismic interface, and mark the various velocity zones.

JRP: 1982

## APPENDIX 1.2 FILL DENSITY TESTS

The physical density tests were carried out according to New Zealand standards. Sand replacement follows N.Z.S. 4402, Part 2P : 1981 Test 17 (A). Balloon densometer follows N.Z.S. 4402, Part 2P : 1981 Test 17 (B). Tube sampling follows N.Z.S. 4402, Part 2P : 1981 Test 17 (C), except that the tube used was 35mm diameter by 150mm long rather than the standard 105mm diameter by 115mm long.

The Troxler nuclear densometer was set up and run, following the Troxler operations manual, by a technician from Soils and Foundations (1973) Ltd, Christchurch.

## APPENDIX 1.3 ROCK AND SOIL DESCRIPTION

Rock and soil descriptions follow the systems of Bell and Pettinga, 1984 (Figure A1.3.1 and A1.3.2 over page).

# ENGINEERING GEOLOGICAL FIELD DESCRIPTION FOR ROCK MATERIAL

## WEATHERING

| | TERM | GRADE | ROCK DESCRIPTION |
|---|---|---|---|
| 1. | residual soil (RW) | VI | discolouration and complete transformation to soil; original fabric destroyed |
| 2. | completely weathered (CW) | V | discolouration and transformation to soil, original fabric largely preserved |
| 3. | highly weathered (HW) | IV | discolouration; discontinuities open and rock fabric affected by deep alteration; lithorelicts present |
| 4. | moderately weathered (MW) | III | discolouration; discontinuities open with discolouration and alteration penetrating inwards; loss of strength |
| 5. | slightly weathered (SW) | II | slight discolouration on open discontinuities; no loss of material strength |
| 6. | unweathered (UW) | I | no discolouration or loss of strength, or any other effects due to weathering |

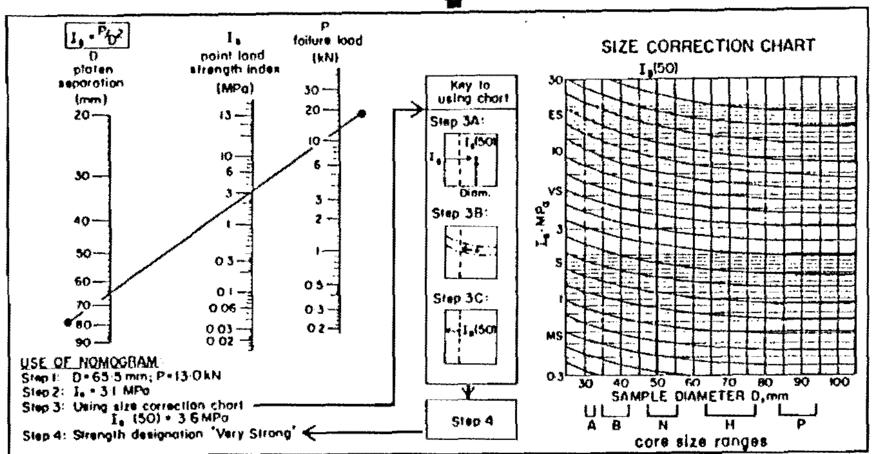
## STRENGTH

| | TERM | POINT LOAD STRENGTH INDEX Is(50) | FIELD ESTIMATION OF STRENGTH |
|---|---|---|---|
| 1. | extremely strong (ES) | more than 10 | can only be chipped with geological hammer |
| 2. | very strong (VS) | 3 to 10 | several blows of hammer required to break hand specimen |
| 3. | strong (S) | 1 to 3 | few firm blows of hammer required to break specimen |
| 4. | moderately strong (MS) | 0·3 to 1 | breaks readily with one blow of hammer |
| 5. | moderately weak (MW) | 0·1 to 0·3 | broken by hand only with difficulty, small thin pieces broken by finger pressure |
| 6. | weak (W) | 0·03 to 0·1 | broken by hand; pieces 25 mm or more broken by finger pressure |
| 7. | very weak (VW) | less than 0·03 | crushed or remoulded by hand (includes hard soils) |

## GEOLOGICAL CLASSIFICATION

| CRYSTAL OR GRAIN SIZE | SEDIMENTARY: CLASTIC | SEDIMENTARY: CHEM/ORGANIC | IGNEOUS: Mode of Occurrence | Silicic | Intermed | Mafic | Ultramafic | METAMORPHIC: FOLIATED | METAMORPHIC: MASSIVE |
|---|---|---|---|---|---|---|---|---|---|
| very coarse 64 | CONGLOMERATE (1) AGGLOMERATE (2) BRECCIA (3) | | INTRUSIVE | (27) (28) GRANITE GRANODIORITE | (29) (30) SYENITE DIORITE | (31) GABBRO | (32) (33) PERIDOTITE DUNITE | GNEISS (34) | HORNFELS (39) |
| coarse 2 | | Calcareous LIMESTONE (9) | | | | | | SCHIST (35) | MARBLE (40) |
| medium | SANDSTONE (4) | Siliceous CHERT (10) OTHER (11) | | | | | | PHYLLITE (36) | |
| 0·06 | | | | | | | | | |
| fine | TUFFS (7); SILTSTONE (5); MUDSTONE (6) | Carbonaceous COAL (12) OTHER (13); Ferruginous LATERITE (14) OTHER (15) | EXTRUSIVE | (19) (20) (21) RHYOLITE OBSIDIAN DACITE | (22) (23) TRACHYTE ANDESITE | (24) (25) DOLERITE BASALT | (26) SERPENTINITE | SLATE (37) | QUARTZITE (41) |
| very fine 0·002 (mm) | CLAYSTONE (8) | Saline ROCK SALT (16) GYPSITE (17) OTHER (18) | | | | | | MYLONITE (38) | AMPHIBOLITE (42) |

| --- WEATHERING TERM | --- STRENGTH TERM | --- --- --- COLOUR | --- FABRIC | --- --- ROCK NAME |
|---|---|---|---|---|




POINT LOAD STRENGTH INDEX

COLOUR

1: light
2: dark

1: pinkish
2: reddish
3: yellowish
4: brownish
5: olive
6: greenish
7: bluish
8: greyish

1: pink
2: red
3: yellow
4: brown
5: olive
6: green
7: blue
8: white
9: grey
0: black

FABRIC

1: finely layered (< 25 mm)
2: coarsely layered (25 - 100 mm)
3: massive
4: other (specify)

Figure A1.3.1 (after Bell & Pettinga, 1984)

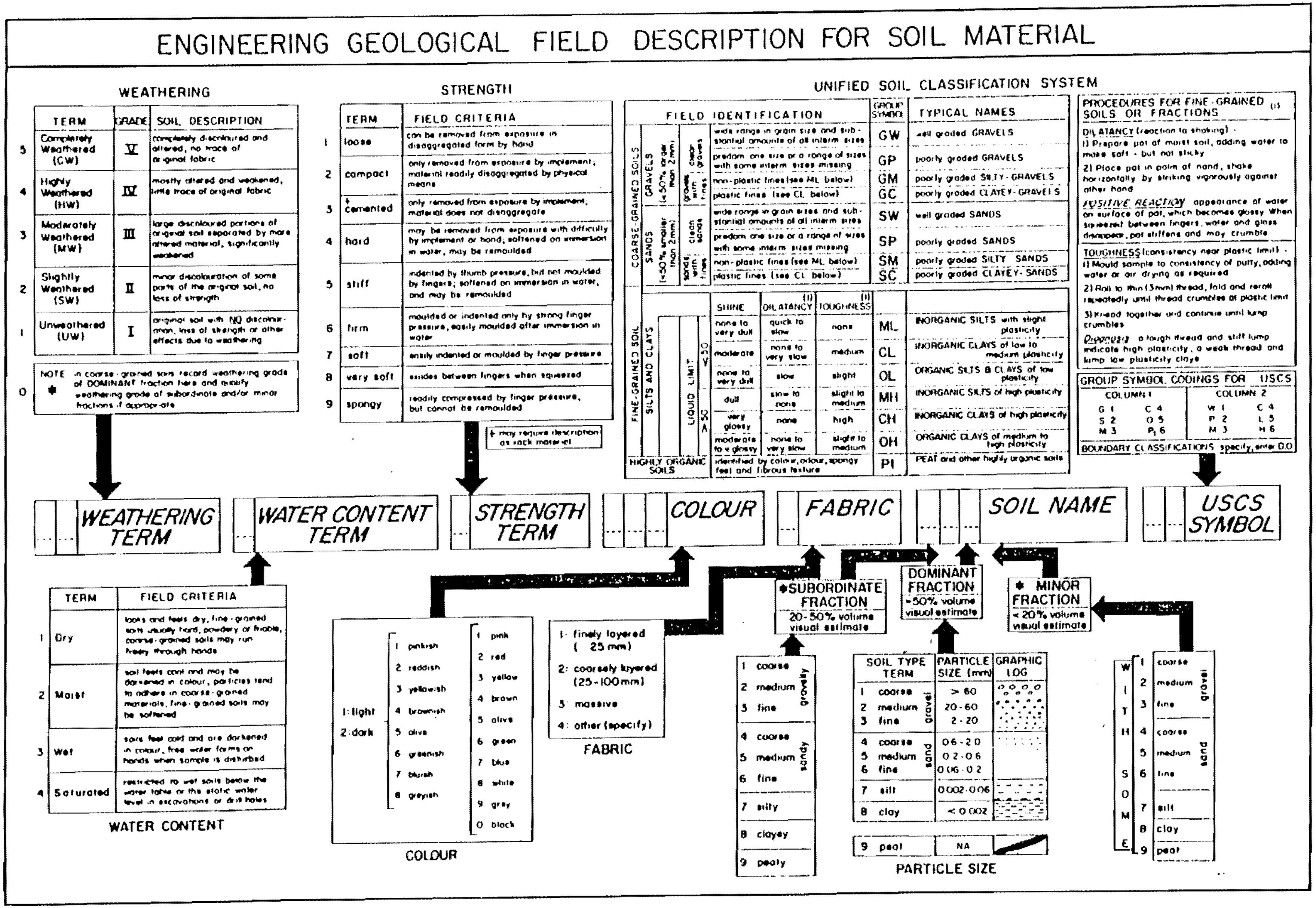



Figure A1.3.2 (after Bell & Pettinga, 1984)

# APPENDIX 2

## INDEX TESTS FOR LOESS

2.1 N.Z. Standard Tests

2.2 Crumb Test

2.3 Pinhole Erosion

APPENDIX 2.1 NEW ZEALAND STANDARD TESTS

Grainsize distributions were determined by wet sieving (N.Z.S. 4402, Part 1 : 1980 test 9 (A)) and the hydrometer method (N.Z.S. 4402 Part 1 : 1980 test 9(D)). Distribution curves are reproduced in Appendix 7. Atterberg limits (i.e. liquid and plastic limits) determination follows N.Z.S. 4402 Part1 : 1980 Tests 2,3,4. Dry density and moisture content determinations follow tube sampling procedures in N.Z.S. 4402, Part 2P : 1981 Test 17(C), but with 35mm diameter by 150mm long thin walled drive tubes.

APPENDIX 2.2 CRUMB TEST

This test is as modified by Yetton (1986). The modifications decrease test time and exclude slaking as a criteria.

Method : a crumb of soil at insitu moisture is dropped into a beaker of distilled water and observed after 10 minutes.

Results : Class 1 no reaction - may slake but no cloudy water.

Class 2 slight reaction - slight cloud in water near surface of crumb.

Class 3 moderate reaction - cloud of colloids in suspension around sample.

Class 4 strong reaction - colloidal cloud virtually obscures the whole bottom of the beaker.

APPENDIX 2.3 PINHOLE EROSION

This test measures the relative erodibilty of samples. The classification has been modified by Yetton (1986) to remove dispersive/non dispersive implications. Water flow rate through the sample is calculated for each minute and erosion is taken to be the head that produces a significant increase in flow rate greater than 0.1ml/sec over a 3 minute period. This erosion value is written as $E_{180}$, where 180 is the head that causes significant erosion. Heads used in the test are 50, 180, 360 and 1000mm, so $E_{50}$ is highly erodible and $E_{>1000}$ is very resistant to erosion.

# APPENDIX 3

## TRIAXIAL TEST METHOD

3.1 Triaxial Method

3.2 Alternative Triaxial Tests

## APPENDIX 3.1 TRIAXIAL METHOD

Tube samples (35mm diameter) were collected, bagged and returned to the laboratory. The samples were trimmed and extruded, inspected for unwanted defects or material heterogeneity, and loaded into a triaxial cell similar to that shown in Figure A3.1.1

Samples were loaded and failed under unconfined, undrained (UU) conditions (Millar, 1982) without pore pressure measurement. Samples of similar moisture content were failed at confining pressures ranging from 50 to 250 kPa in increments of 20 to 50 kPa. Displacement versus axial load was plotted by a chart recorder for each sample.

Failure was taken as the peak load, or at 10% axial strain of the sample. failure loads and confining pressures convert to P-Q points, which are plotted together for samples of similar moisture content. A $k_f$ line is fitted to the points by eye, as recommended by Fell and Jeffery (1987). The $k_f$ line for a P-Q plot is converted to a total stress failure envelope according to the relationship given in Figure A3.1.2. Total stress parameters ($c,\phi$) are calculated from the ($a,\alpha$) values of the $k_f$ line. P-Q plots for this study are reproduced in Appendix 4.

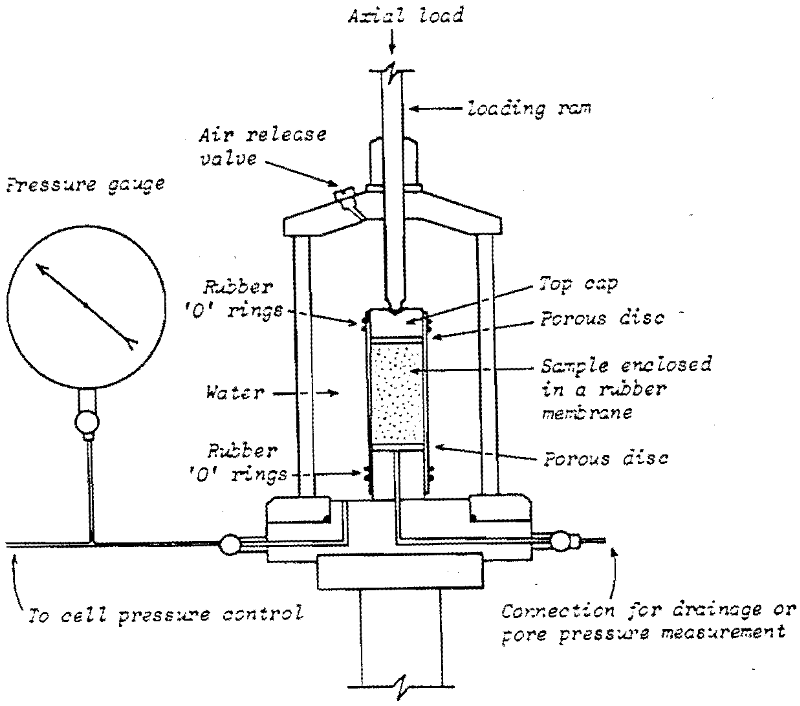
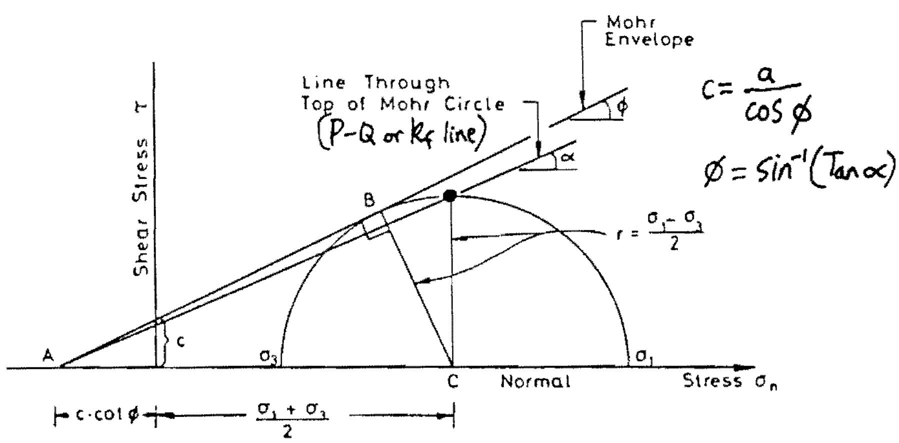



Figure A3.1.1 Diagramatic Layout of the Triaxial Test (after Millar, 1982)

Figure A3.1.2 Basis of P-Q plot (modified after Fell & Jeffery, 1987)



APPENDIX 3.2 ALTERNATIVE STRENGTH TESTS.

Pells etal (1973) have compared direct shear and triaxial methods and found no significant difference in results for silt materials, from North America, at low to moderate (700 kPa) confining pressures. Pardanyi & Vago (1973) obtained similar results from tests run on Hungarian loess. In any case direct shear is most useful for measuring strength of defects. Material strength was being measured for Coleridge Tce samples, so the relative ease and speed of collecting and running triaxial samples gives the method an advantage over direct shear. The reasons for not using other triaxial test methods are outlined in the following paragraphs.

Results from a UU test cannot be analysed for effective stress parameters (i.e. $c', \phi'$). The complexities of obtaining effective stress parameters from a partially saturated silt (e.g. loess) are such that it is well outside the time frame of a masters thesis project and may require several years of research dedicated specifically to strength testing. Partial saturation and low permeability ($10^{-7}$ to $10^{-9}$ m/sec) of loess are the properties that complicate the strength testing process.

Undrained triaxial tests with pore pressure measurement are an accepted way of obtaining effective stress parameters from saturated low permeability materials (Millar, 1982). However, pore pressure of partially saturated soils consists of pore water and pore air components, with the basic relationship (after Lambe & Whitman, 1979) $u^* = u_a a_a + u_w a_w$

where $u^*$ is an equivalent pore pressure

$u_a$ is pore air pressure

$u_w$ is pore water pressure

$a_w$ factor related to degree of saturation

The effective stress equation $\sigma' = \sigma - u$ becomes

$$\sigma' = \sigma - u_a + a_w(u_a - u_w)$$

Lambe & Whitman (1979) note that special techniques and advanced equipment are needed to measure the parameters mentioned above, as well as there being doubt over how $a_w$ should be determined. They continue, suggesting that -"the best procedure to estimate strength is to run tests that duplicate the field conditions as closely as possible: same degree of saturation, same total stress and, if possible, the same pressure in the liquid phase."

A quick (loaded 1mm/min) UU test, as used in this project (see Appendix 1.3), could be carried out on the available equipment and also meets the criteria of Lambe & Whitman quoted above. The only problem is the requirement to have similar pressure in the liquid phase for lab test and field condition. There is the possibility that quick failure of the sample could lead to rapid rises in pore pressure that do not model conditions existing during failure of a natural slope, i.e. the slope fails in a drained condition. However, it is just as likely that rapid increases in pore pressure could be a contributing factor to slope failure, in which case quick UU tests are modeling field conditions. Loading rates slower than 1mm/min (tending towards a drained condition) were not tried, because samples and lab time were dedicated to comparing strengths at different moisture contents (attempted duplication of field conditions), requiring the consistent use of one test method. Some samples were failed with pore pressure monitoring to try and gain an idea of pressure fluctuations in the sample, but due to low permeability of the sample it is doubtful that pore pressure at the end of the sample adequately reflects pore pressure along the failure plane.

Drained triaxial tests of saturated material directly measure effective stress parameters. A sample is loaded at a rate slow enough, so as not to cause a rise in pore pressure, therefore increasing load is taken as effective stress on the soil skeleton (Millar, 1982). The problem with low permeability materials, such as loess, is the time taken for equalisation of excess pore pressures. Samples would have to be loaded in small increments then left until pore pressures dissipated. This process would require several weeks careful monitoring for each sample, with any operator errors or malfunctions of equipment invalidating the test. Samples would have to be saturated by back pressure and consolidated at the required confining pressure, once again the low permeability means each sample would take at least a week to prepare for testing. It may not be possible to saturate samples of loess, even from the highest field moistures encountered (approx. 14-15%, degree of saturation approx 0.7), as Millar (1982) states that a degree of saturation of at least 0.9 is required to achieve satisfactory results with back pressure saturation in the standard triaxial apparatus. If saturation could be achieved it would still take several months to test a suite of 4 or 5 samples.

Partially saturated samples could be tested under drained conditions, but the same time constraints as for saturated drained tests would apply. This type of test would be truly experimental (no references found in the literature) and some check on results would be essential. Testing suites of samples at different moisture contents would perhaps provide a check on the nature of strength parameters being measured (i.e. effective stress, total stress, something else?). As mentioned earlier in this section, investigating alternative methods of strength testing can rapidly develop into a research topic in its own right.

# APPENDIX 4

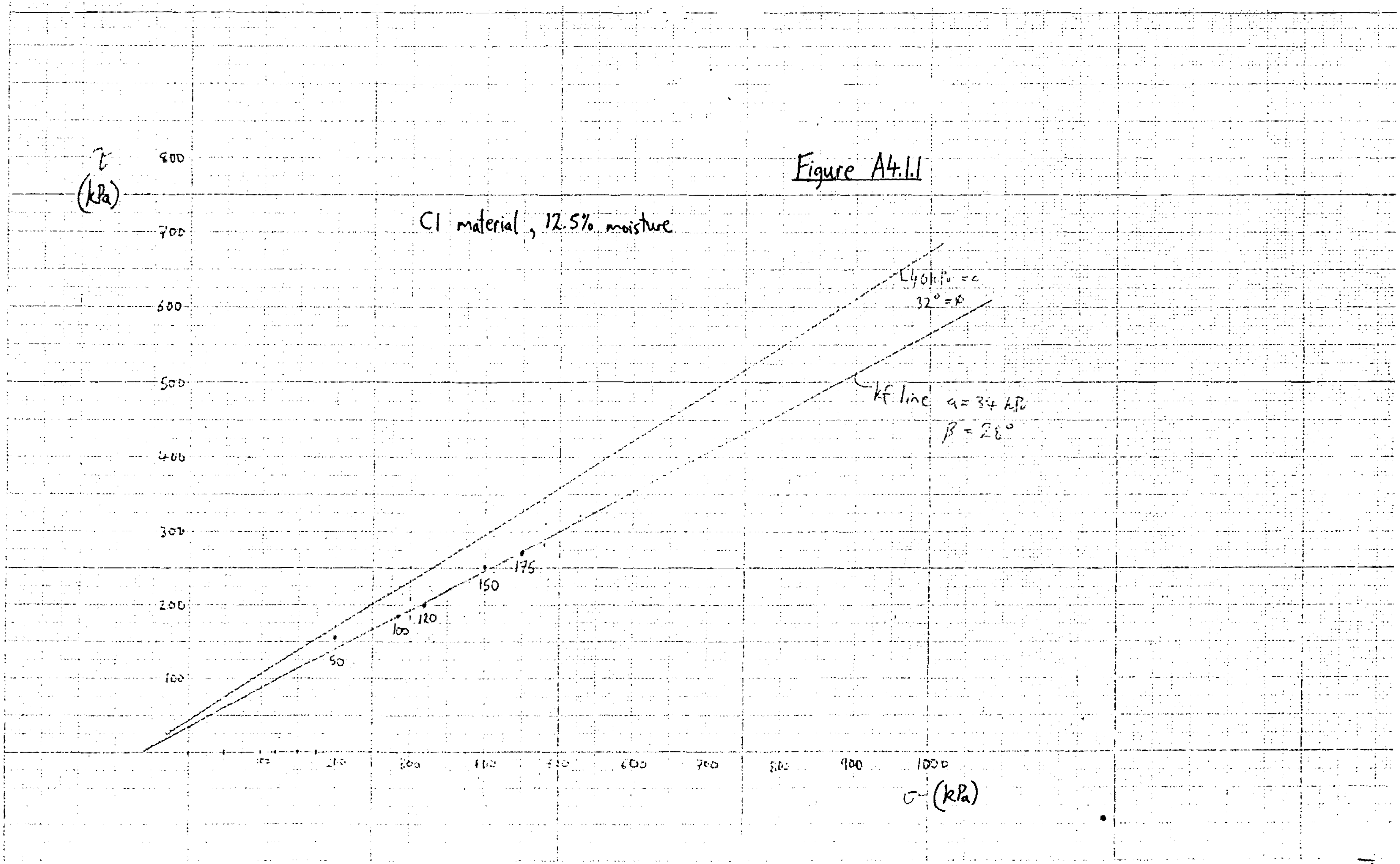
## P-Q PLOTS FOR TRIAXIAL TESTS

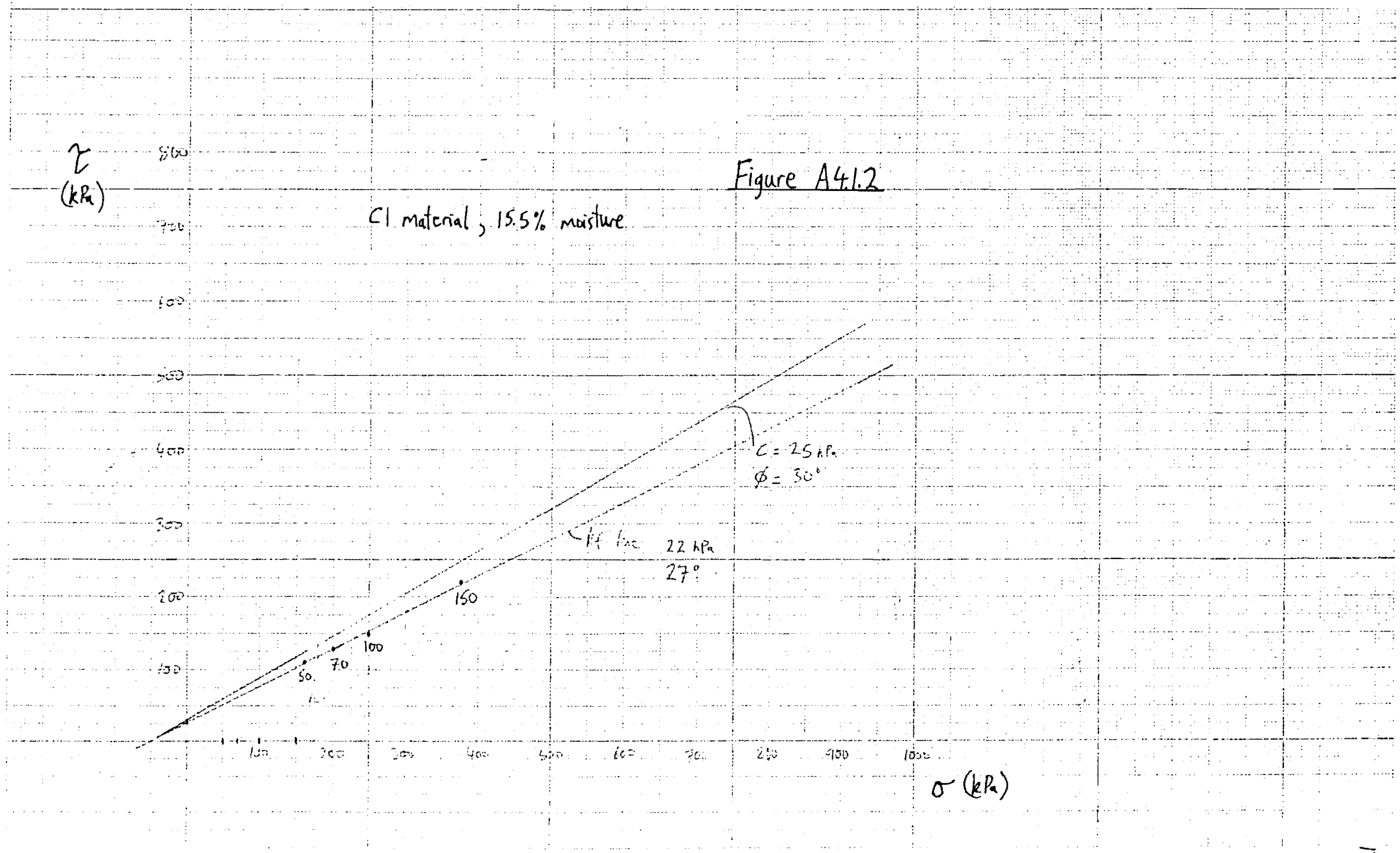
4.1 Coleridge Tce Results

4.2 Westmorland Results

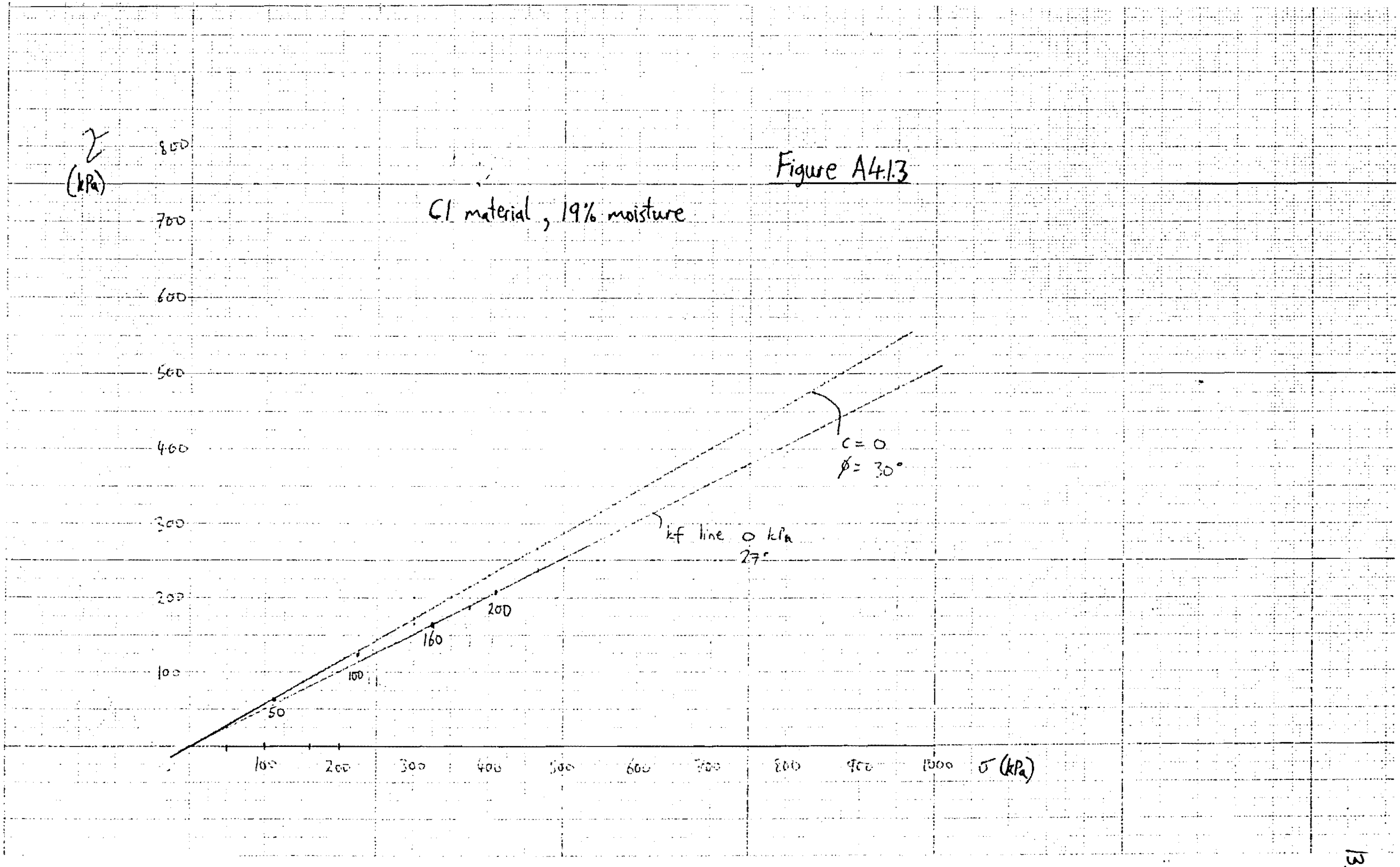
| Sample number | W% | $\sigma_3$ (kPa) | $\sigma_1$ (kPa) | P $\left(\frac{\sigma_1+\sigma_3}{2}\right)$ | Q $\left(\frac{\sigma_1-\sigma_3}{2}\right)$ | C (kPa) | $\phi$ (°) | a (kPa) | $\alpha$ (°) |
|---|---|---|---|---|---|---|---|---|---|
| C1 12.5% | 12.9 | 50 | 361 | 205 | 155 | 40 | 32 | 34 | 28 |
| | 13.1 | 100 | 470 | 285 | 185 | | | | |
| | 12.8 | 120 | 520 | 320 | 200 | | | | |
| | 12.4 | 150 | 650 | 400 | 250 | | | | |
| | 12.6 | 175 | 724 | 450 | 275 | | | | |
| C1 15.5% | 16.1 | 50 | 273 | 161 | 111 | 25 | 30 | 22 | 27 |
| | 16.2 | 70 | 329 | 199 | 129 | | | | |
| | 15.5 | 100 | 397 | 248 | 148 | | | | |
| | 15.3 | 150 | 590 | 370 | 220 | | | | |
| C1 19% | 19.1 | 50 | 175 | 112 | 62 | 0 | 30 | 0 | 27 |
| | 18.6 | 100 | 350 | 225 | 125 | | | | |
| | 19.4 | 160 | 490 | 325 | 165 | | | | |
| | 19.5 | 200 | 618 | 409 | 209 | | | | |
| C2 9% | 8.9 | 50 | 700 | 375 | 325 | 140 | 33 | 117 | 28.5 |
| | 8.8 | 75 | 772 | 423 | 348 | | | | |
| | 8.5 | 100 | 970 | 535 | 435 | | | | |
| | 8.9 | 150 | 960 | 555 | 405 | | | | |
| C2 11.5% | 10.9 | 50 | 560 | 305 | 255 | 90 | 34 | 75 | 29 |
| | 11.0 | 85 | 701 | 393 | 308 | | | | |
| | 11.5 | 130 | 730 | 430 | 300 | | | | |
| | 11.8 | 150 | 875 | 512 | 363 | | | | |
| | 11.1 | 200 | 1000 | 600 | 400 | | | | |
| C2 12.5% | 12.5 | 50 | 390 | 220 | 170 | 40 | 36 | 32 | 30.5 |
| | 12.4 | 100 | 550 | 325 | 225 | | | | |
| | 12.4 | 120 | 630 | 375 | 255 | | | | |
| | 12.0 | 150 | 780 | 465 | 315 | | | | |
| | 12.3 | 175 | 905 | 540 | 365 | | | | |
| | 12.9 | 200 | 982 | 591 | 391 | | | | |
| C2 15.5% | 15.5 | 50 | 280 | 165 | 115 | 20 | 32 | 17 | 28 |
| | 15.4 | 100 | 420 | 260 | 160 | | | | |
| | 15.9 | 150 | 598 | 374 | 224 | | | | |
| | 15.6 | 200 | 748 | 474 | 274 | | | | |

Table A4.1.1 Triaxial Data for Coleridge Tce Samples


τ (kPa)
800
700
600
500
400
300
200
100
Cl material, 12.5% moisture
40 kPa = c
32° = φ
kf line a = 34 kPa
β = 28°
50
100
120
150
175
600
700
900
1000
σ (kPa)
Figure A4.1.1

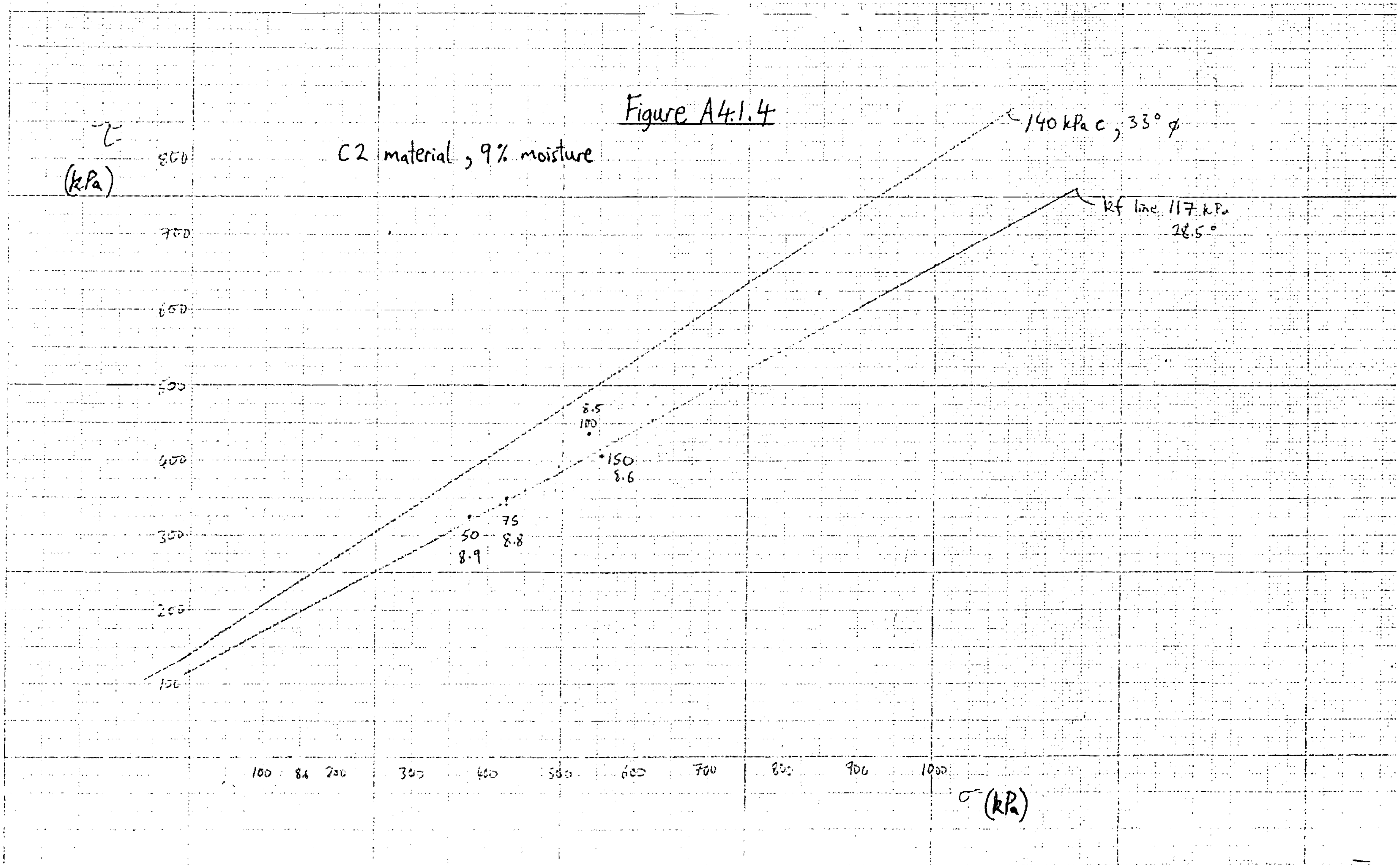


τ (kPa)
800
700
600
500
400
300
200
100
Figure A4.1.2
Cl material, 15.5% moisture
c = 25 kPa
φ = 30°
Kf line
22 kPa
27°
150
100
70
50.
100
200
300
400
500
600
700
800
900
1000
σ (kPa)



τ (kPa)
800
700
600
500
400
300
200
100
Cl material, 19% moisture
c = 0
ϕ = 30°
kf line 0 kPa
27°
50
100
160
200
100
200
300
400
500
600
700
800
900
1000
σ (kPa)


Figure A4.1.3



τ (kPa)
800
700
600
500
400
300
200
100
C2 material, 9% moisture
140 kPa c, 33° φ
Rf line 117 kPa 28.5°
8.5
100
150
8.6
75
8.8
50
8.9
100
8.6
200
300
400
500
600
700
800
900
1000
σ (kPa)


Figure A4.1.4

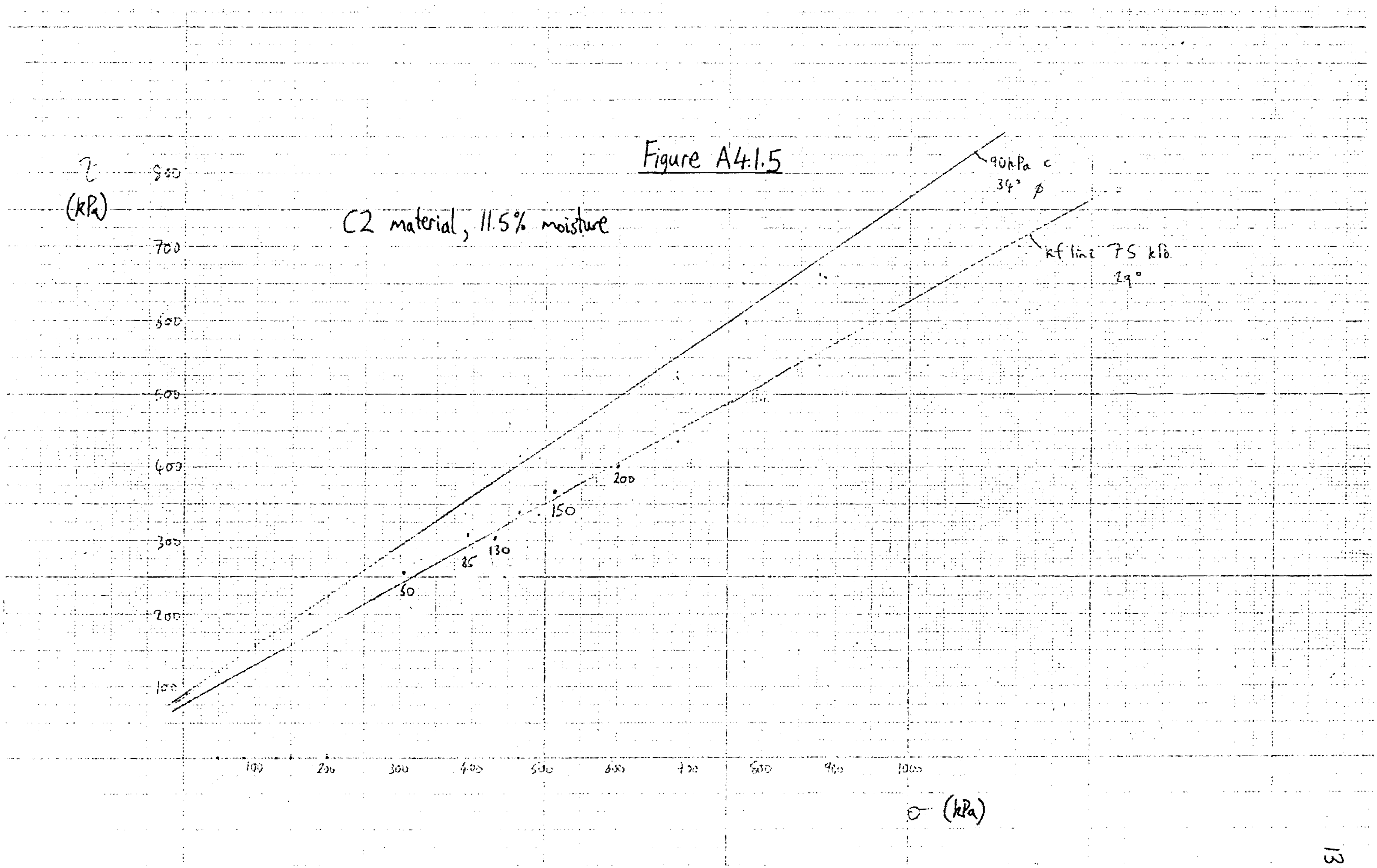
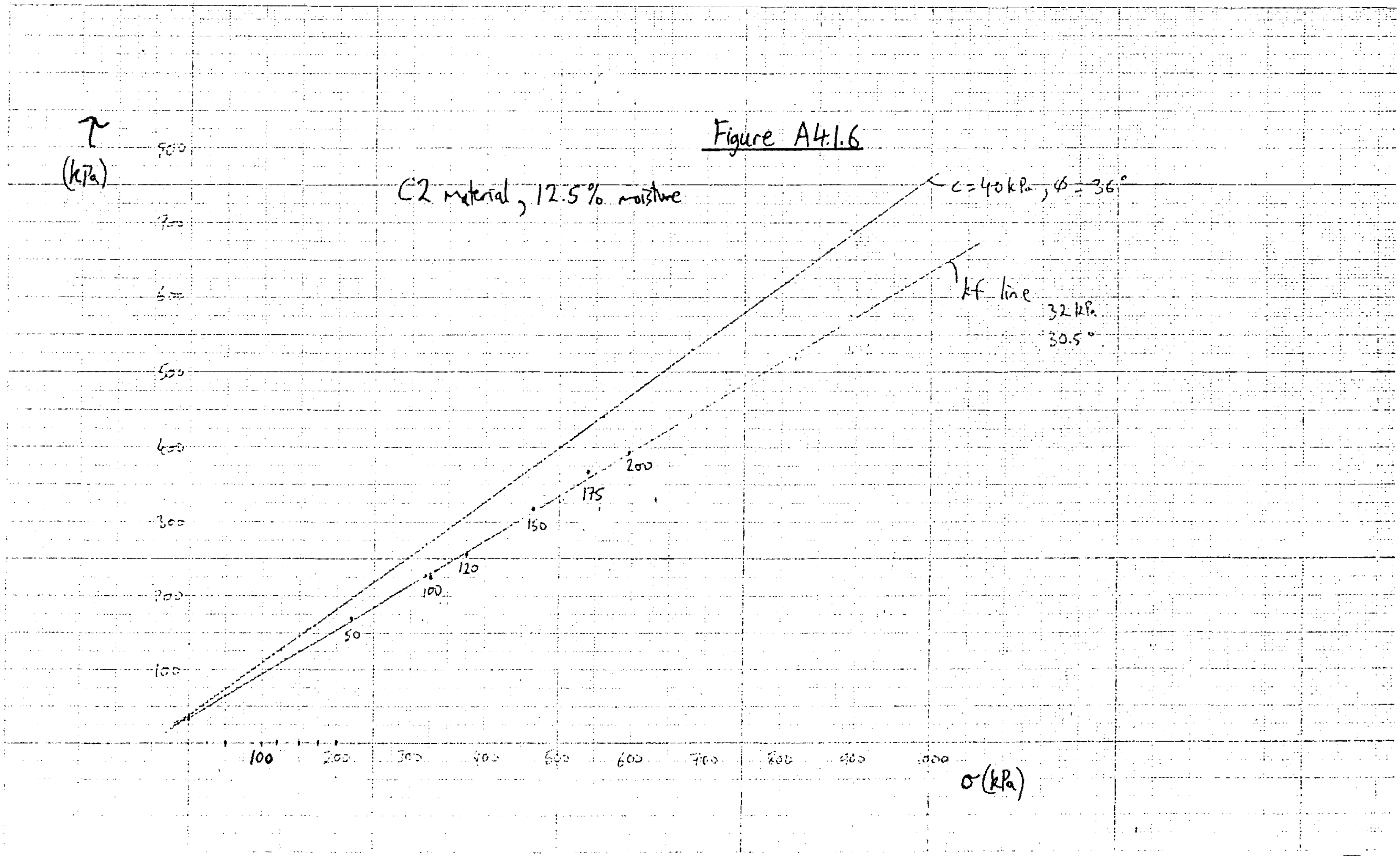


Figure A4.1.5
C2 material, 11.5% moisture
τ (kPa)
800
700
600
500
400
300
200
100
100
200
300
400
500
600
700
800
900
1000
σ (kPa)
90 kPa c
34° φ
Rf line 75 kPa
29°
50
85
130
150
200

## Figure A4.1.6



τ (kPa)
C2 material, 12.5% moisture
c = 40 kPa, φ = 36°
kf line
32 kPa
30.5°
50
100
120
150
175
200
100
200
300
400
500
600
700
800
900
1000
σ (kPa)

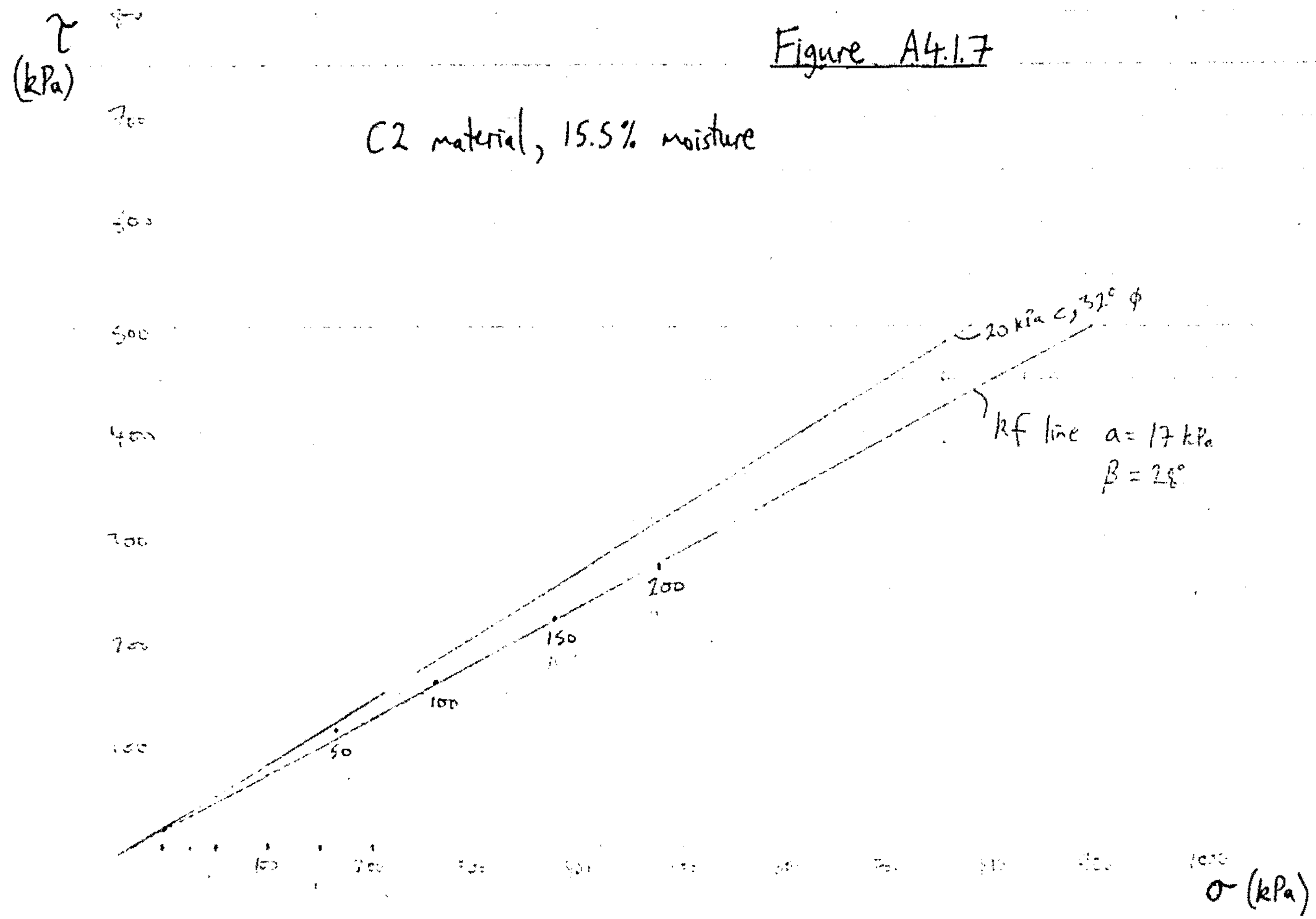

τ (kPa)
C2 material, 15.5% moisture
20 kPa c, 32° φ
kf line a = 17 kPa
β = 28°
50
100
150
200
σ (kPa)

Figure A4.1.7

Figure A4.2.1 P-Q Plot and Mohr Circles for Cut1 (Westmorland)

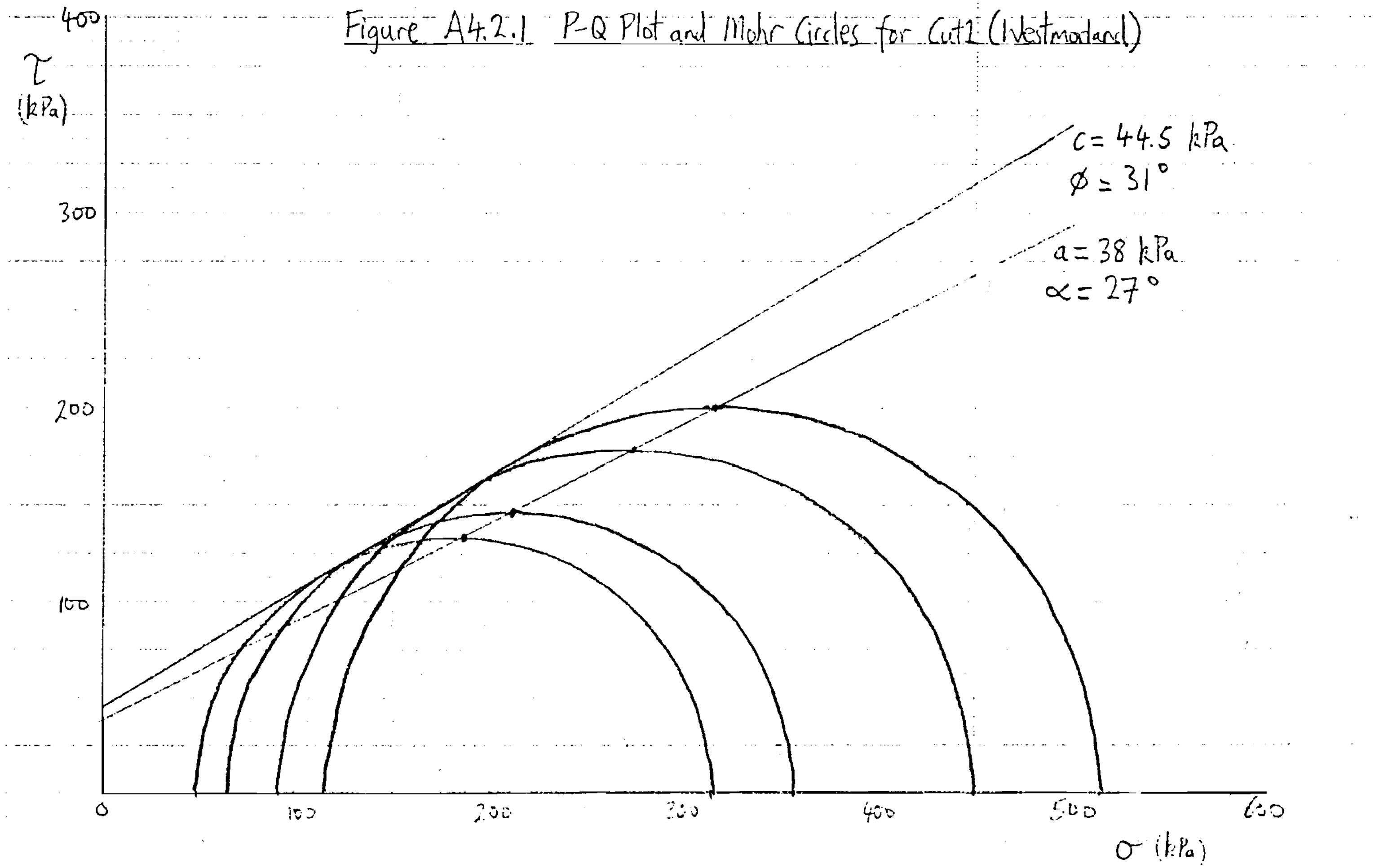


Figure A4.2.2 P-Q Plot and Mohr Circles for Fill 1 (Westmorland)

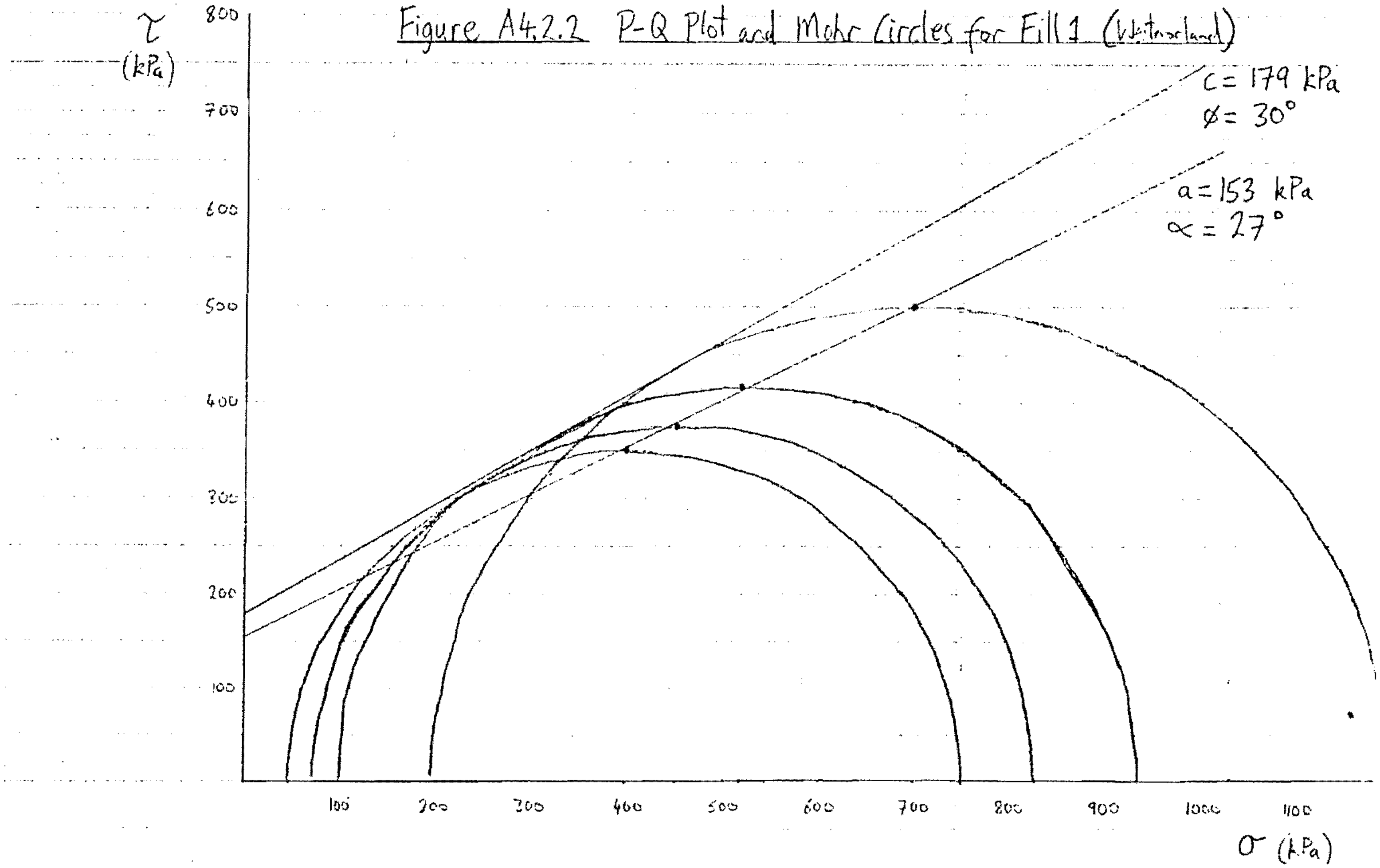
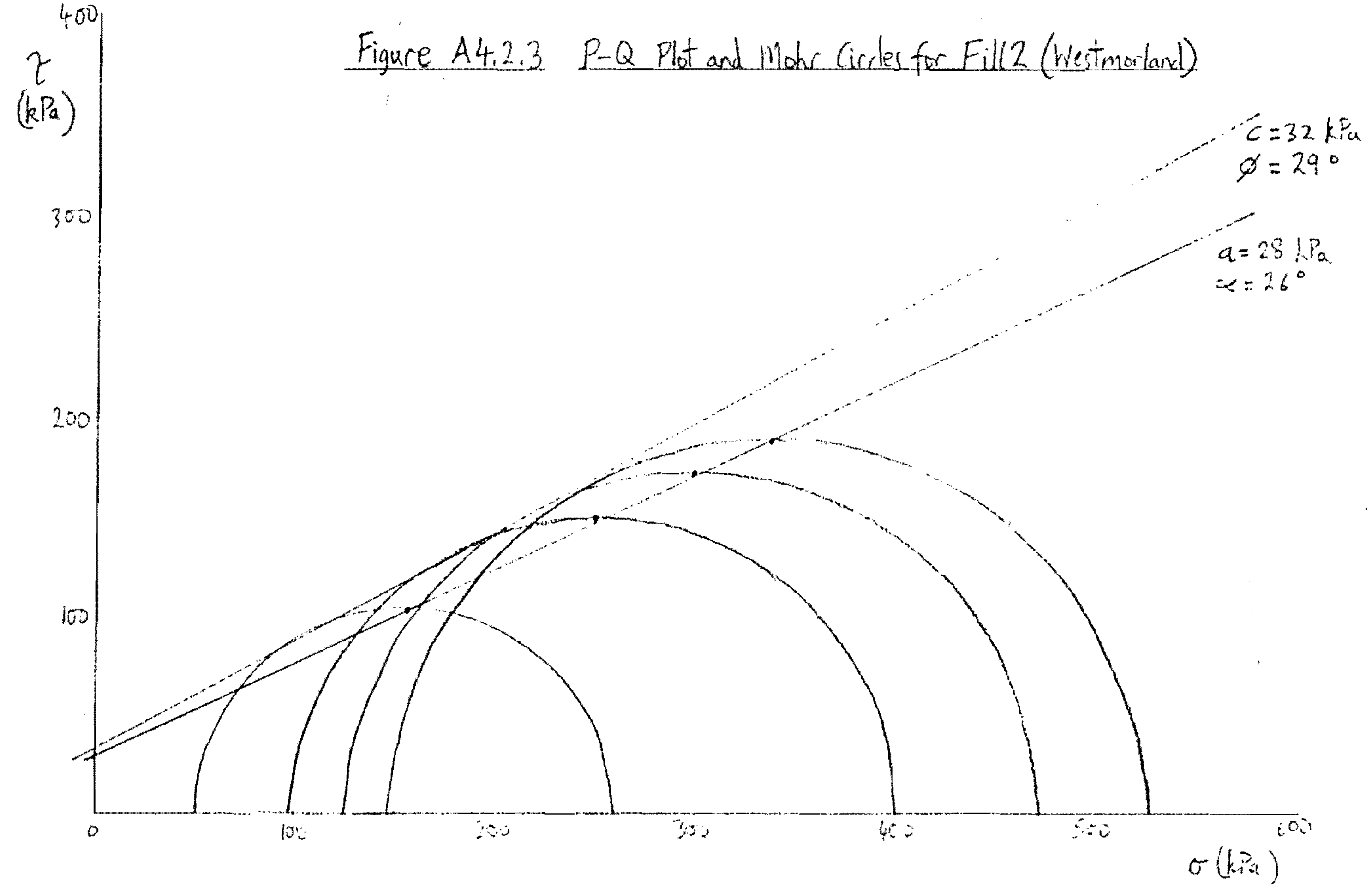


Figure A4.2.3 P-Q Plot and Mohr Circles for Fill2 (Westmorland)



# APPENDIX 5
# RESULTS OF FILL DENSITY TESTS

| Date | Site | Nuclear Densometer Dry Density | Nuclear Densometer W% | Nuclear Densometer Bulk Density | Tube Samples Dry Density | Tube Samples W% | Tube Samples Bulk Density |
|---|---|---|---|---|---|---|---|
| 5/2/88 | 1 | 1660 | 15.9 | 1920 | 1880 | 11.2 | 2090 |
| | 2 | 1710 | 13.3 | 1940 | 1860 | 10.3 | 2050 |
| | 3 | 1720 | 13.5 | 1950 | 1820 | 10.5 | 2010 |
| | 4 | 1670 | 17.2 | 1960 | 1720 | 12.1 | 1930 |
| | 5 | 1670 | 14.3 | 1910 | 1770 | 10.9 | 1960 |
| 24/2/88 | 1 | 1680 | 10.5 | 1860 | 1800 | 9.5 | 1970 |
| | 2 | 1620 | 13.4 | 1830 | 1810 | 9.7 | 1990 |
| | 3 | 1730 | 11.7 | 1930 | 1860 | 10.3 | 2050 |
| | 4 | 1590 | 11.2 | 1670 | 1690 | 9.1 | 1840 |
| | 5 | 1510 | 14.5 | 1730 | 1730 | 9.8 | 1900 |
| 29/2/88 | 1 | 1800 | 14.8 | 2070 | 1930 | 11.5 | 2150 |
| | 2 | 1500 | 8.0 | 1620 | 1550 | 6.6 | 1650 |
| | 3 | 1550 | 10.1 | 1700 | 1660 | 8.5 | 1800 |
| | 4 | 1640 | 10.2 | 1820 | 1780 | 8.0 | 1920 |
| | 5 | 1650 | 8.7 | 1800 | 1720 | 7.8 | 1850 |
| 28/3/88 | 1 | 1660 | 10.1 | 1830 | 1740 | 8.4 | 1890 |
| | 2 | 1660 | 11.2 | 1850 | 1740 | 8.6 | 1890 |
| | 3 | 1710 | 10.8 | 1890 | 1800 | 10.4 | 1990 |
| 3/5/88 | 1 | 1720 | 9.6 | 1880 | 1800 | 10.4 | 1990 |
| 17/5/88 | 1 | 1650 | 11.4 | 1840 | 1820 | 9.5 | 1990 |
| | 2 | 1630 | 9.8 | 1790 | 1760 | 8.2 | 1900 |
| | 3 | 1600 | 10.1 | 1770 | 1740 | 8.7 | 1890 |
| | 4 | 1680 | 10.3 | 1860 | 1860 | 8.7 | 2020 |

Table A5.1 Fill Density Data (ND & tube)

| Date | Site | Tube Samples Dry Density | W% | Bulk Density | Balloon Densometer Dry Density | W% | Bulk Density | Sand Replacement Dry Density | W% | Bulk Density |
|---|---|---|---|---|---|---|---|---|---|---|
| 5/3/88 | 1 | 1870 | 8.8 | 2030 | 1750 | 9.2 | 1910 | - | - | - |
| | 2 | 1650 | 6.5 | 1760 | 1610 | 6.6 | 1720 | - | - | - |
| | 3 | 1820 | 9.9 | 2000 | 1880 | 8.7 | 2040 | - | - | - |
| 14/5/88 | 1 | 1810 | 9.2 | 1980 | 1780 | 9.0 | 1940 | 1760 | 8.9 | 1920 |
| | 2 | 1760 | 8.5 | 1910 | 1800 | 8.8 | 1960 | 1780 | 8.5 | 1930 |
| | 3 | 1850 | 9.8 | 2030 | 1810 | 9.7 | 1990 | 1880 | 9.5 | 2060 |
| | 4 | 1780 | 8.6 | 1930 | 1720 | 8.1 | 1860 | 1700 | 8.6 | 1850 |
| | 5 | 1720 | 8.7 | 1870 | 1750 | 8.4 | 1900 | 1780 | 8.9 | 1940 |
| | 6 | 1880 | 9.4 | 2060 | 1790 | 9.6 | 1960 | 1820 | 9.4 | 1990 |
| | 7 | 1700 | 8.2 | 1840 | 1730 | 8.3 | 1870 | 1750 | 8.1 | 1890 |
| | 8 | 1780 | 9.4 | 1950 | 1720 | 9.2 | 1880 | 1700 | 9.3 | 1860 |

Table A5.2 Fill Density test Data (physical tests)

# APPENDIX 6

## WESTMORLAND CUT RESULTS

6.1 Cut Face Logs

6.2 Index Test Results

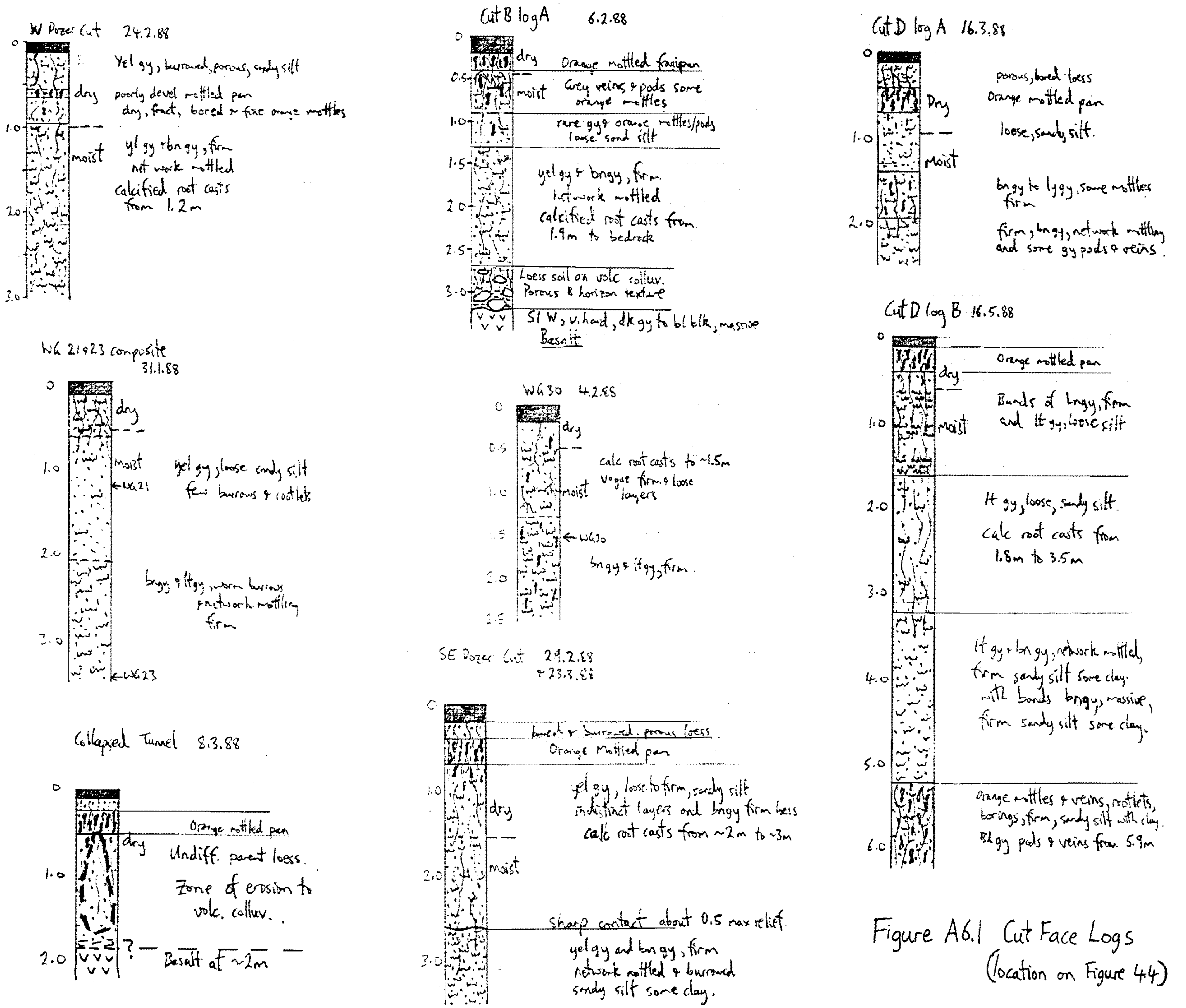



Figure A6.1 Cut Face Logs
(location on Figure 44)

| Sample | Dry density (kg/m³) | Insitu W% | Atterberg Limits WL | WP | WI | Grain size (%) clay | silt | sand | Crumb (class) | Pinhole Erosion | Loess layer |
|---|---|---|---|---|---|---|---|---|---|---|---|
| WG4 | 1670 | 19.4 | 24 | 19 | 5 | 15 | 71 | 14 | 1 | – | Loess colluvium |
| WG7 | 1620 | 5.7 | 2 | 20 | 5 | 15 | 72 | 13 | 2 | E180 | weathered loess |
| WG10 | 1600 | 7.0 | 23 | 18 | 5 | 16 | 65 | 19 | 3 | E180 | network mottled |
| WG21 | 1480 | 12.1 | N. P. | | | 10 | 75 | 15 | 2 | E50 | airfall loess |
| WG23 | 1580 | 13.3 | N. P. | | | 14 | 69 | 17 | 2 | E180 | airfall loess |
| WG25 | 1650 | 21.8 | 24 | 20 | 4 | 13 | 66 | 21 | 1 | E360 | loess colluvium |
| WG27 | 1640 | 6.2 | 25 | 18 | 7 | 17 | 70 | 13 | 2 | E360 | weathered loess |
| WG30 | 1630 | 9.0 | 22 | 20 | 2 | 13 | 71 | 16 | 2 | E360 to E1000 | Buried soil |
| WG31 | 1720 | 13.0 | 22 | 19 | 3 | 17 | 71 | 12 | 2 | – | Buried soil |
| WG32 | 1670 | 15.1 | 23 | 19 | 4 | 15 | 73 | 12 | 2 | E360 | Buried Soil |
| WG33 | – | – | 66 | 31 | 35 | 45 | 28 | 27 | 1 | – | Highly weathered Basalt. |
| CD1 | 1580 | 4.6 | 21 | 18 | 3 | 17 | 72 | 11 | 1 | – | Weathered loess |
| CD2 | 1750 | 9.5 | 27 | 18 | 9 | 26 | 65 | 9 | 1 | E>1000 | Upper fragipan |
| CD3 | 1410 | 7.4 | N. P. | | | 8 | 78 | 14 | 2 | E50 | Airfall loess |
| CD4 | 1630 | 8.5 | 23 | 19 | 4 | 13 | 74 | 13 | 2 | E360 | Buried Soil |
| CD6 | 1410 | 9.6 | – | – | – | – | – | – | 2 | E50 to E180 | airfall loess |
| CD7 | 1620 | 13.5 | – | – | – | – | – | – | 2 | E180 | network mottled |
| CD8 | 1850 | 14.3 | – | – | – | – | – | – | 1 | E>1000 | Buried fragipan |
| Cut 1 | 1720 | 13.5 | 25 | 18 | 7 | 17 | 72 | 11 | 2 | E1000 | Buried Soil/ fragipan |
| Fill 1 | 1800 | 8.5 | 23 | 19 | 4 | 15 | 73 | 12 | 2 | E360 to E1000 | Recompacted fill |
| Fill 2 | 1650 | 8.0 | 23 | 19 | 4 | – | – | – | 2 | E180 to E360 | Recompacted fill |

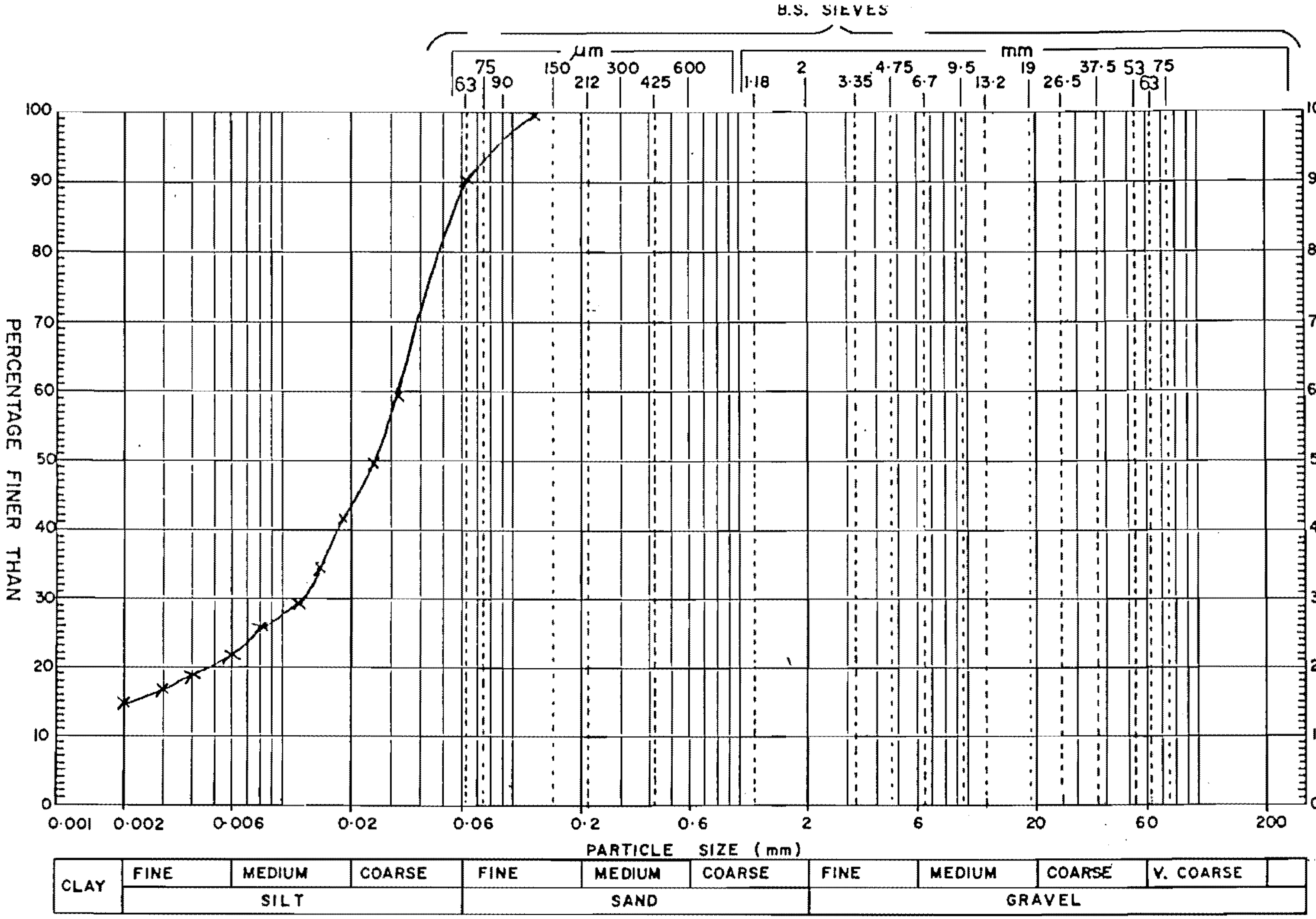
Table A6.2 Index Test Results for Westmorland.

# APPENDIX 7

## GRAINSIZE DISTRIBUTION CURVES

7.1 Coleridge Tce

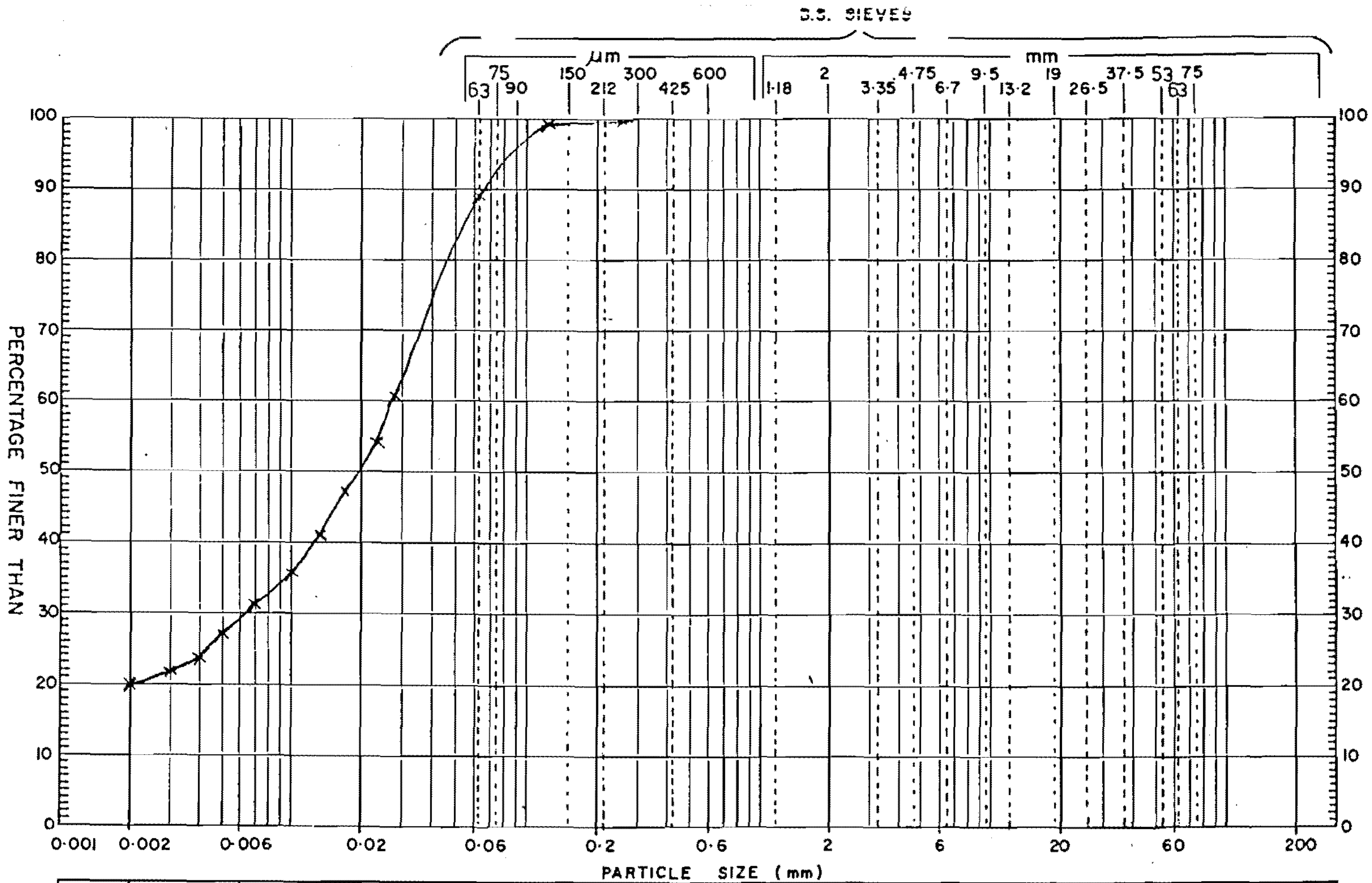
7.2 Westmorland




| | | | |
|---|---|---|---|
| JOB: *Coleridge Tce* | SAMPLE No. *L2* | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | ** | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

*Figure A7.1.1*

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

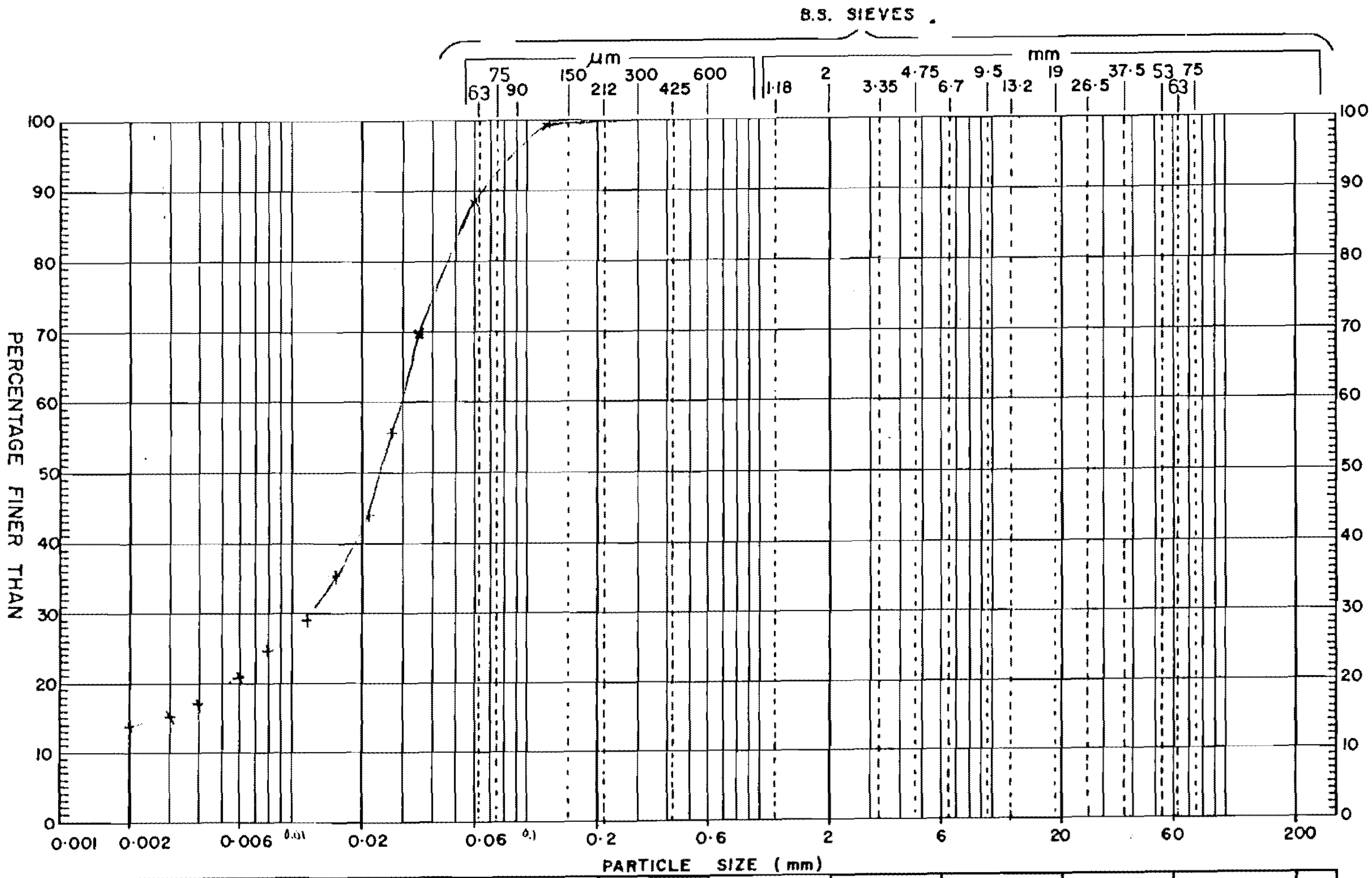



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| JOB: Coleridge Tce | SAMPLE No. L3 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
|---|---|---|---|
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.1.2

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

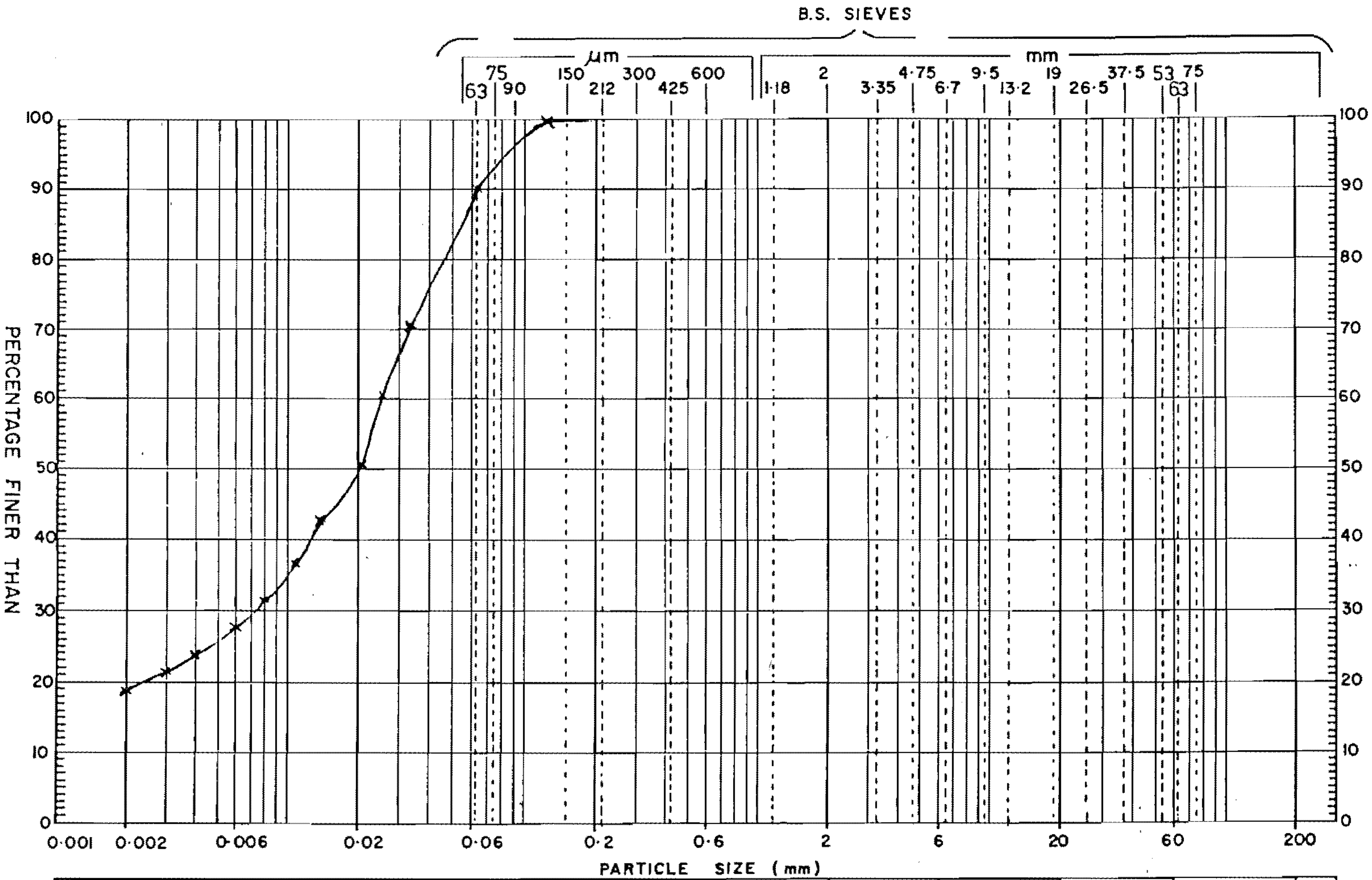



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: Coleridge Tce | SAMPLE No. C1 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.1.3

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

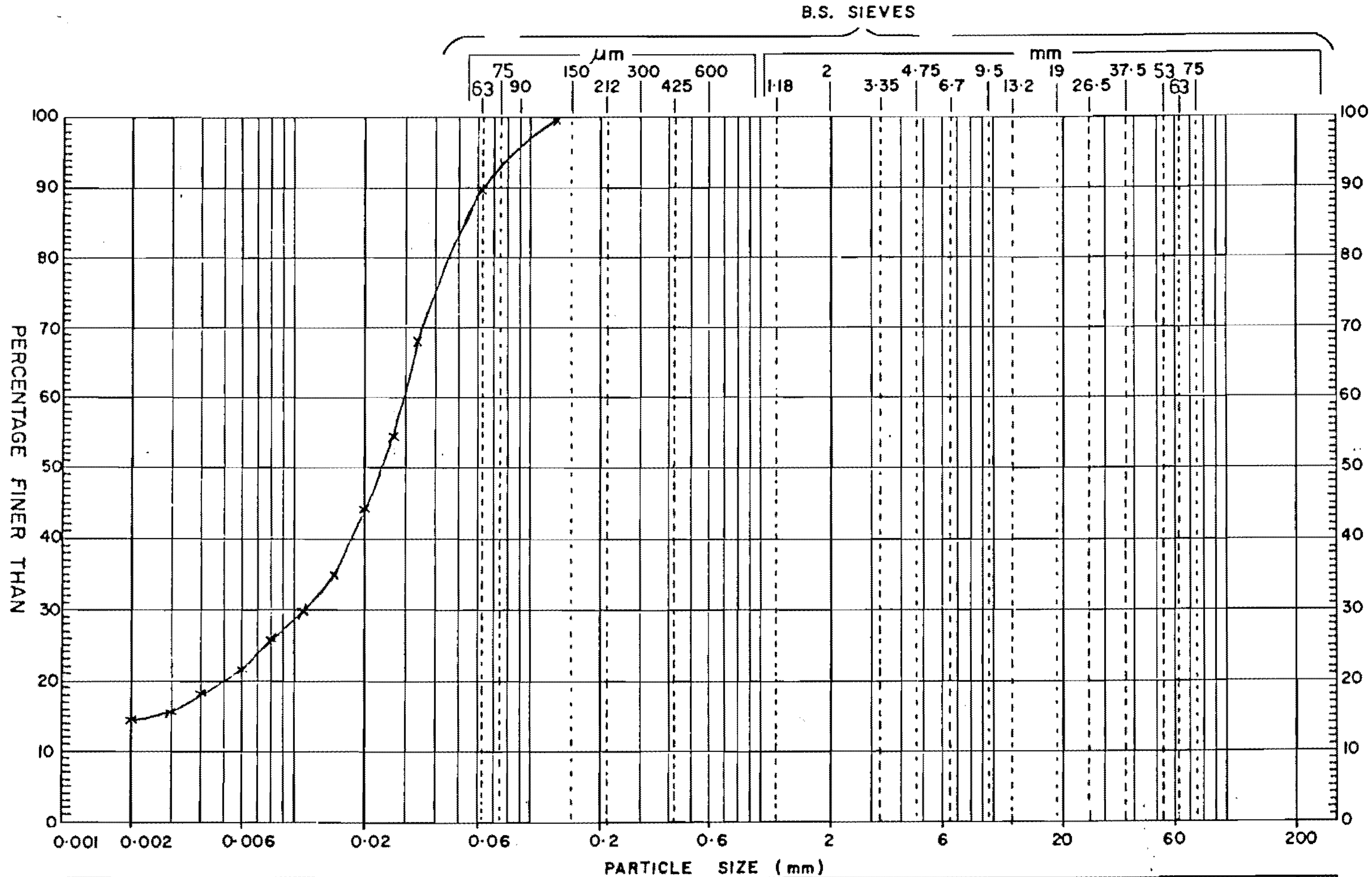



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: Coleridge Tce | SAMPLE No. C2 | NATURAL / AIR DRIED / OVEN DRIED / UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.1.4

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

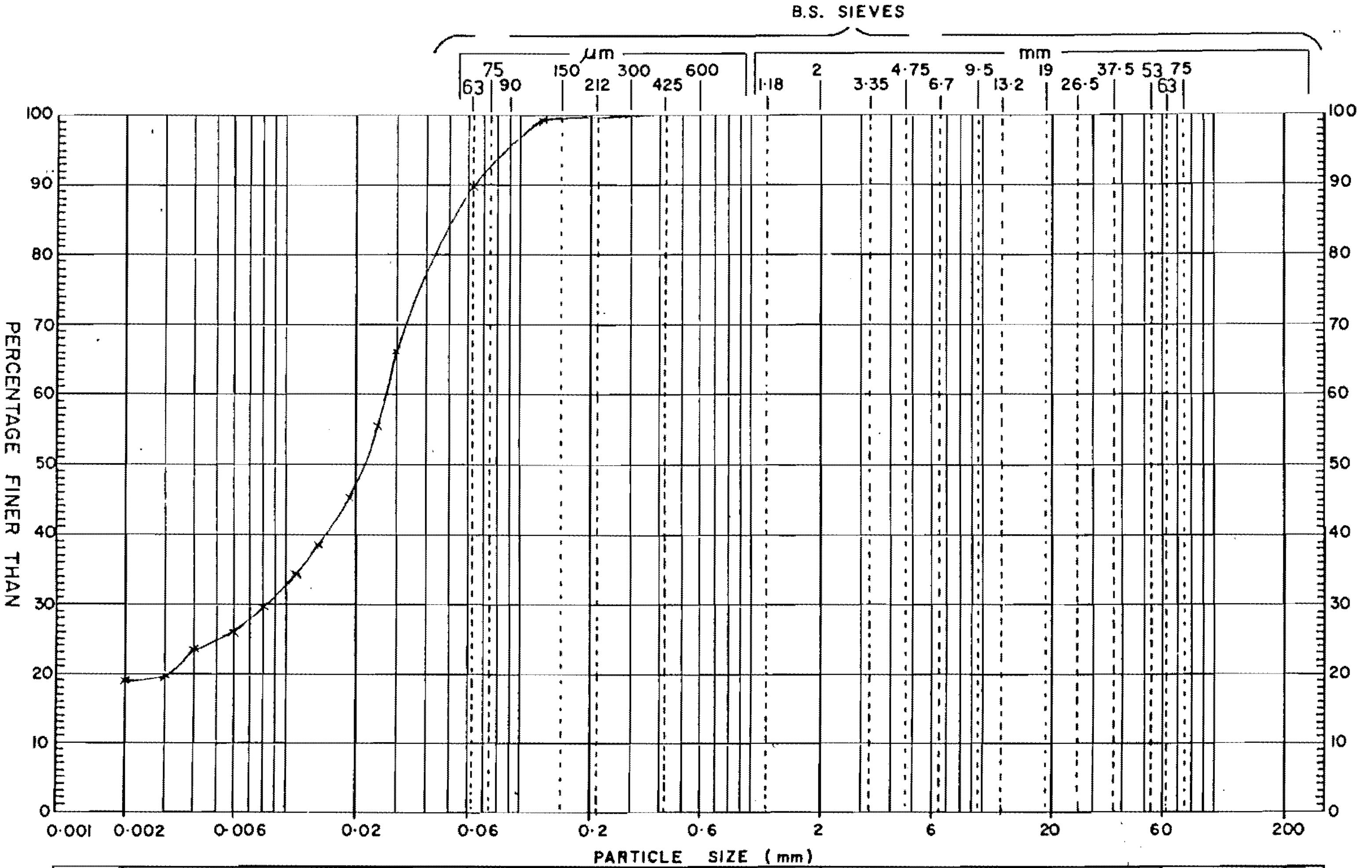



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: Coleridge Tce | SAMPLE No. C3 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.1.5

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

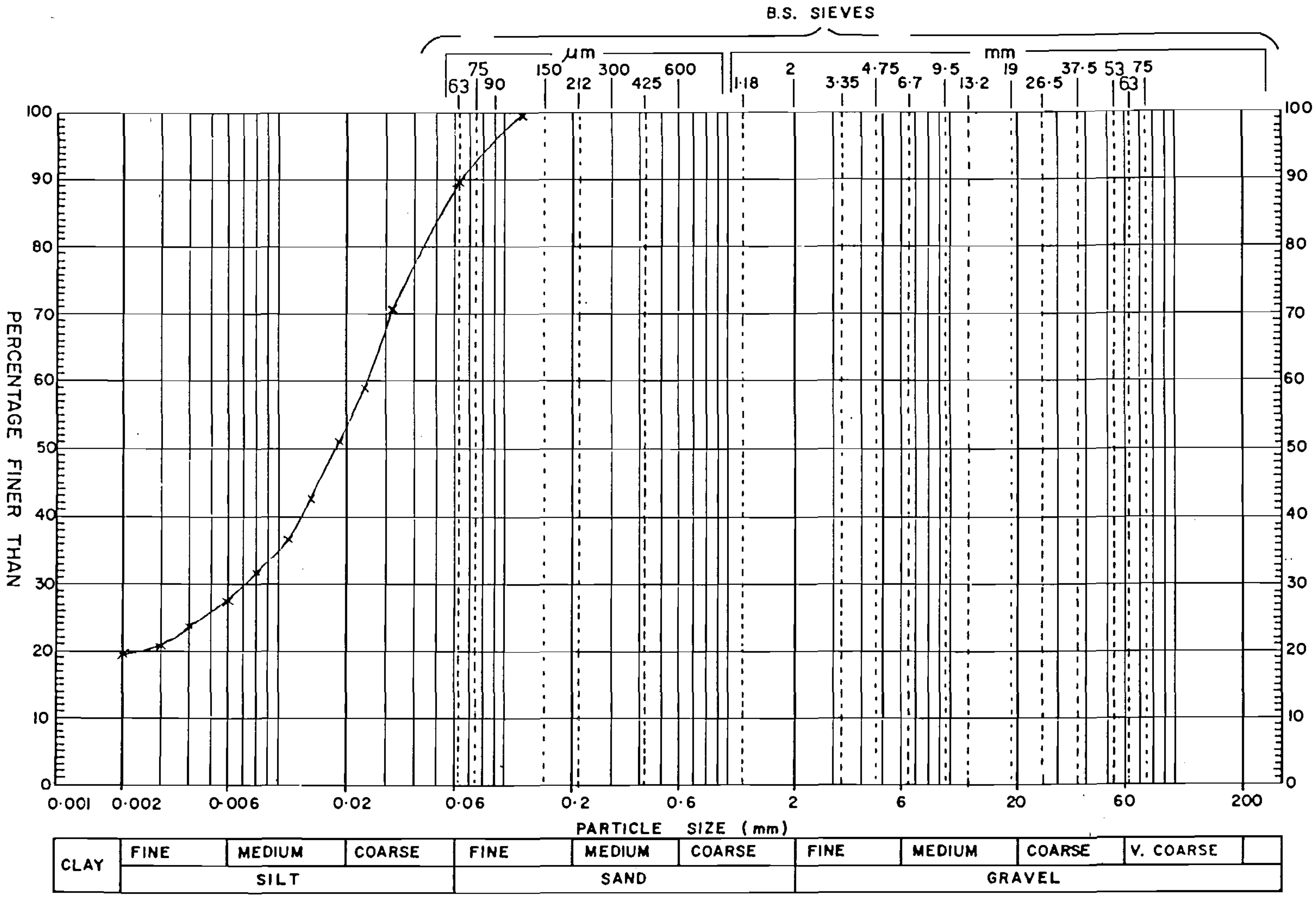



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: Coleridge Tce | SAMPLE No. C4 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | \ | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A.7.1.6

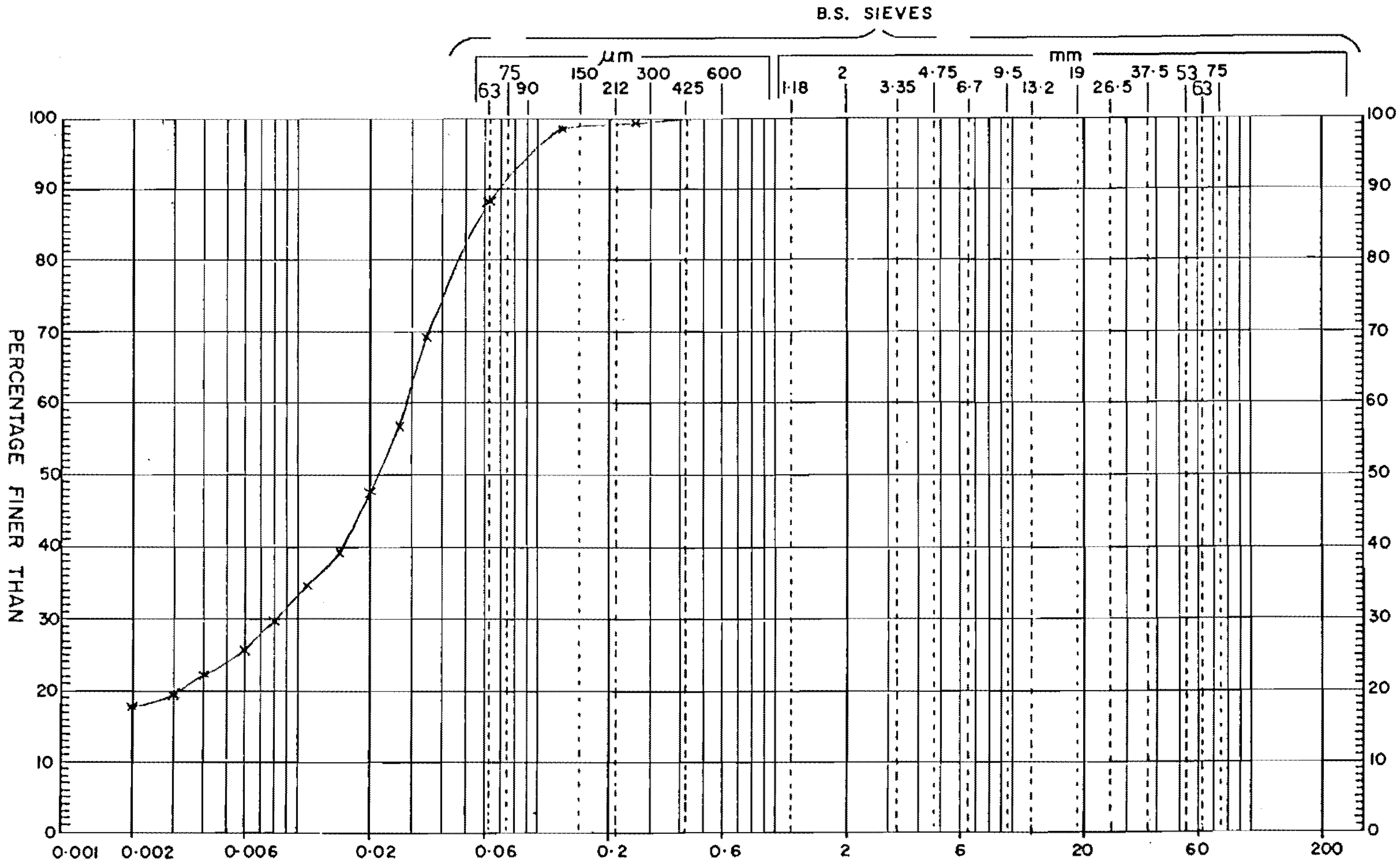
CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION




| | | | |
|---|---|---|---|
| JOB: Coleridge Tce | SAMPLE No. C5 | NATURAL/AIR DRIED/OVEN DRIED/ UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | \ | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.1.7

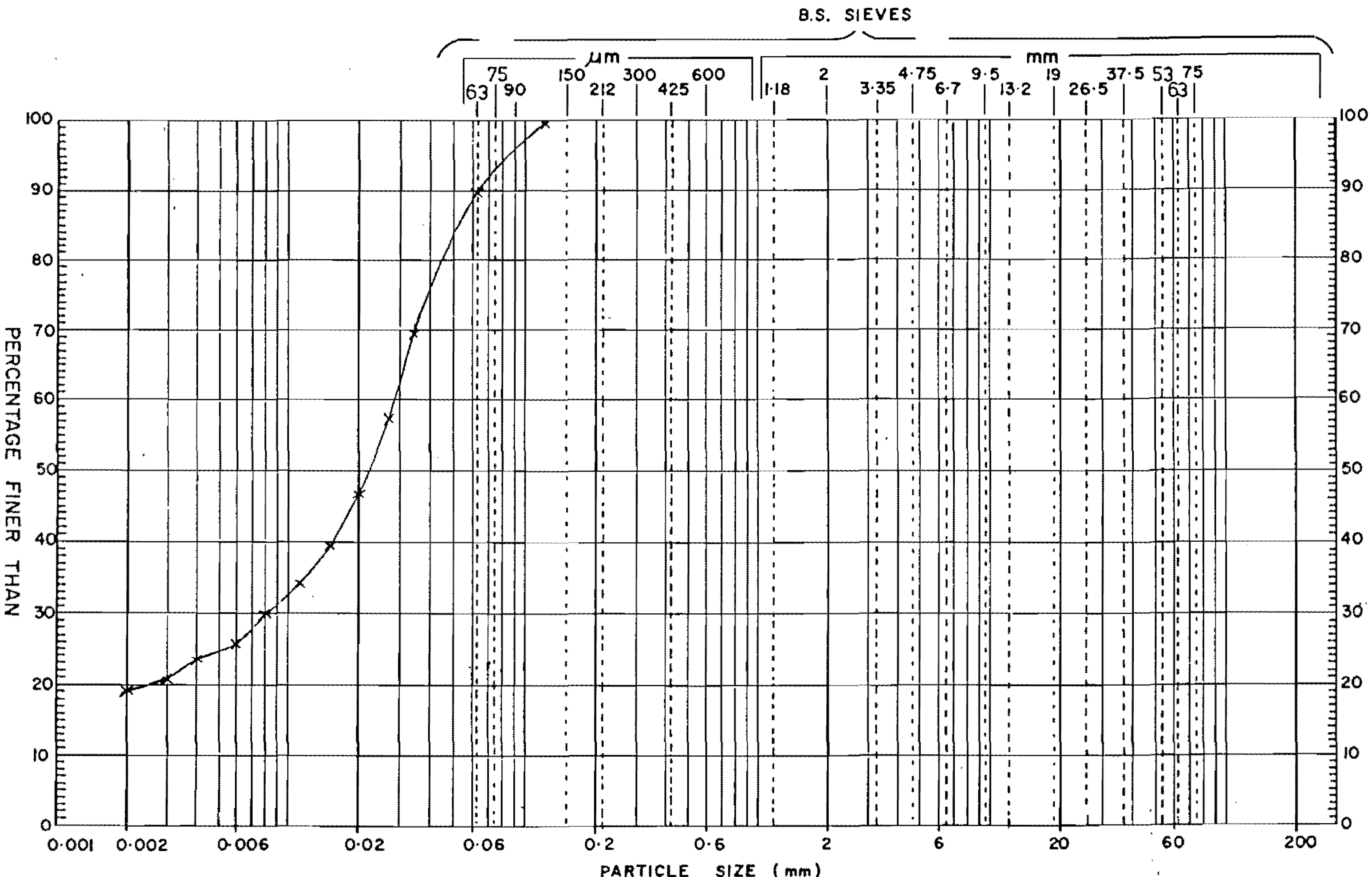
CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION




| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | | |

| | | | |
|---|---|---|---|
| JOB: Coleridge Tce | SAMPLE No. C6 | NATURAL / AIR DRIED / OVEN DRIED / UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | \ | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.1.8 CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

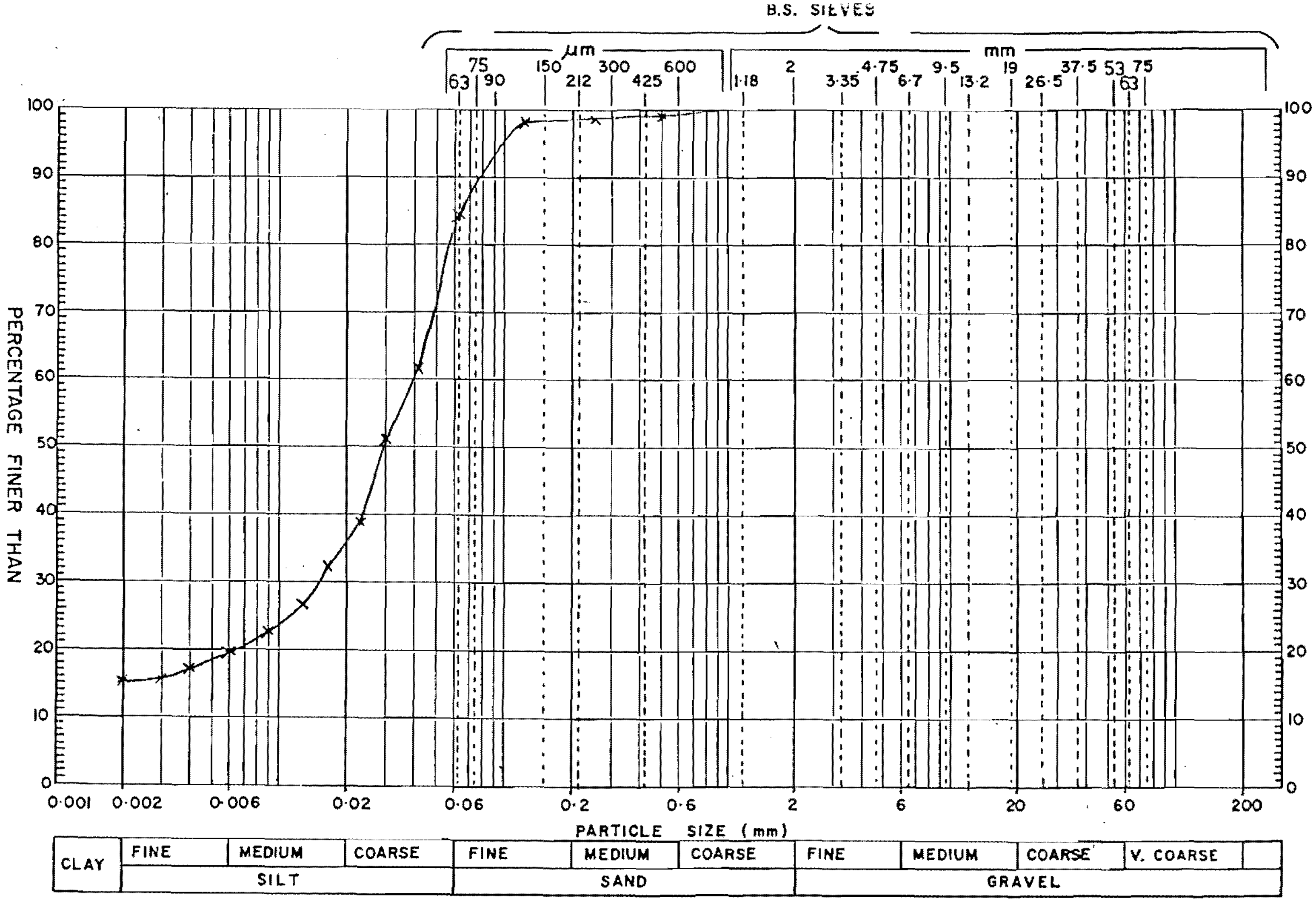



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: Coleridge Tce | SAMPLE No. C7 | NATURAL / AIR DRIED / OVEN DRIED / UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | \ | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.1.9

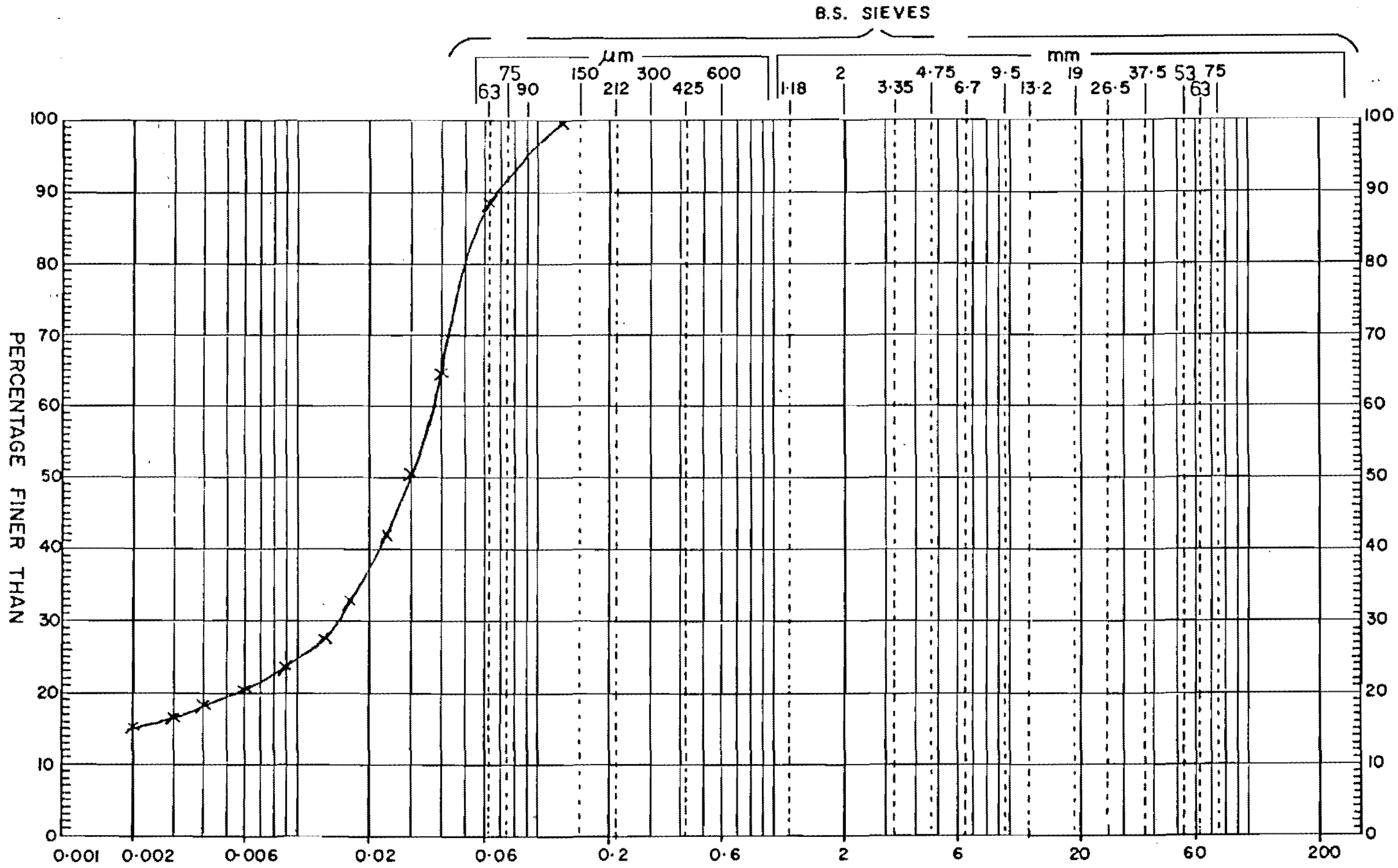
CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION




| | | | |
|---|---|---|---|
| JOB: [illegible] | SAMPLE No. WG4 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | \ | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.1

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION



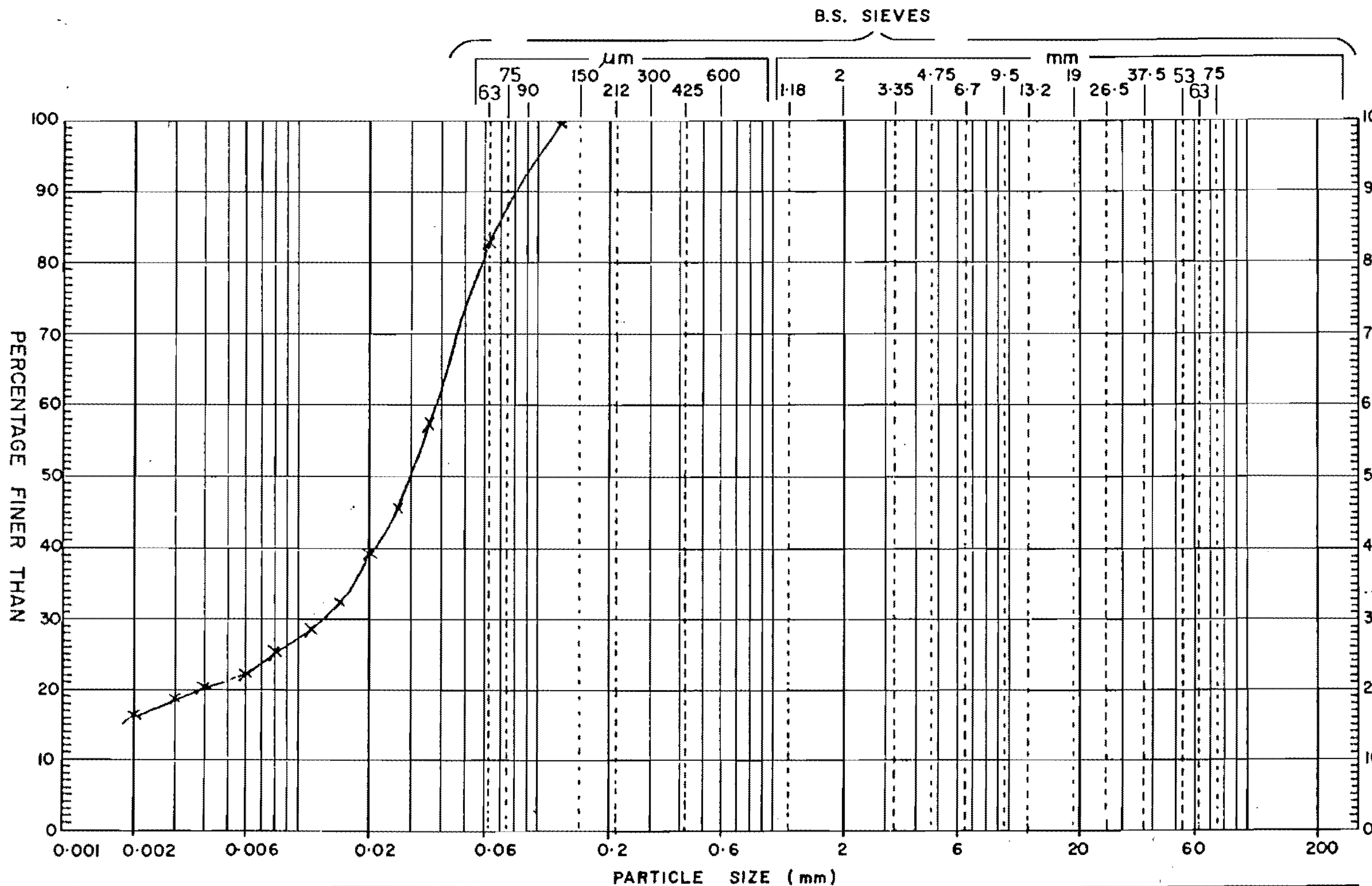

| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: Westmorland | SAMPLE No. WG7 | NATURAL/AIR DRIED/OVEN DRIED/ UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | \ | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.2

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

160

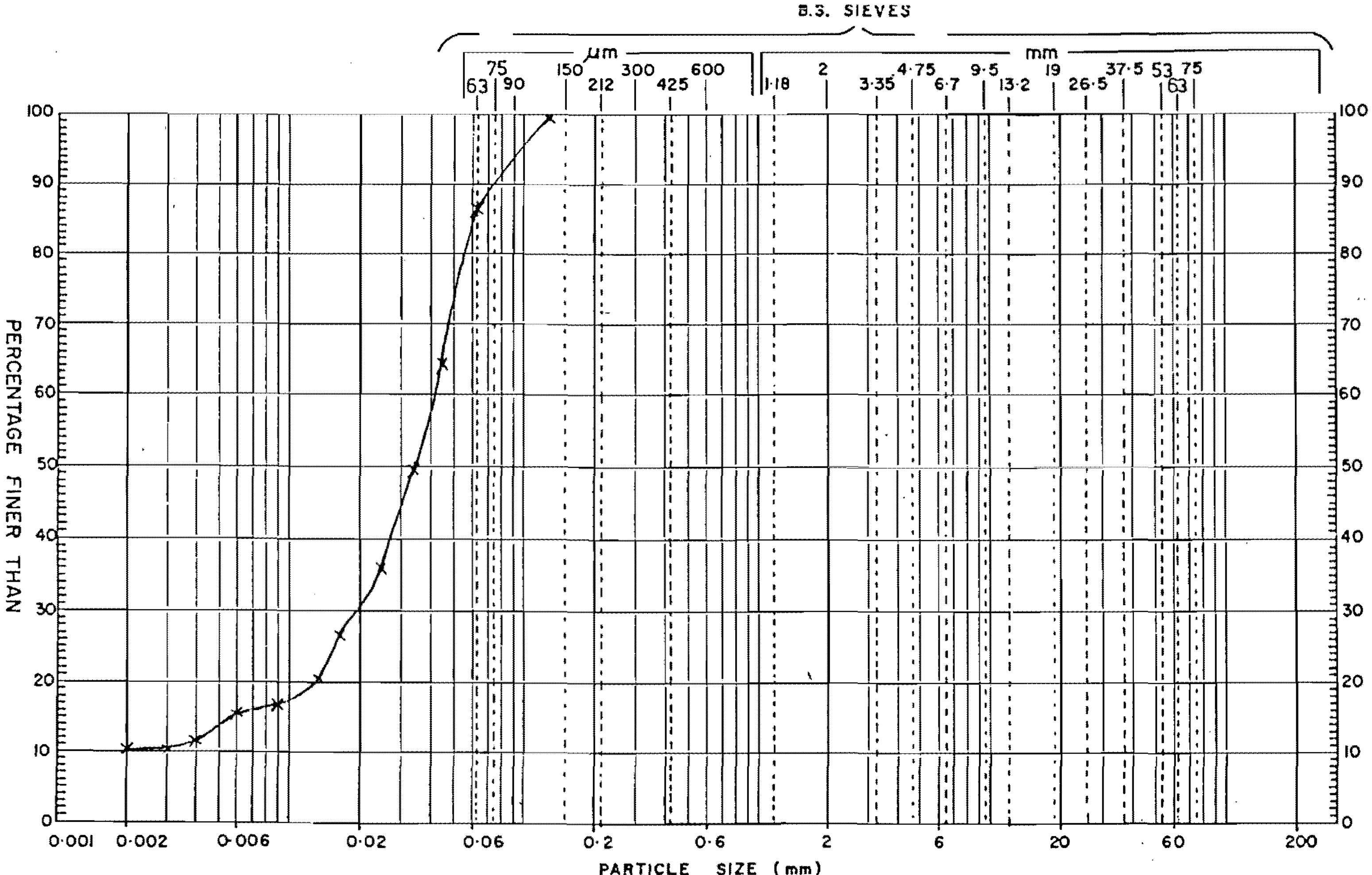



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: [illegible] | SAMPLE No. WG10 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.3

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

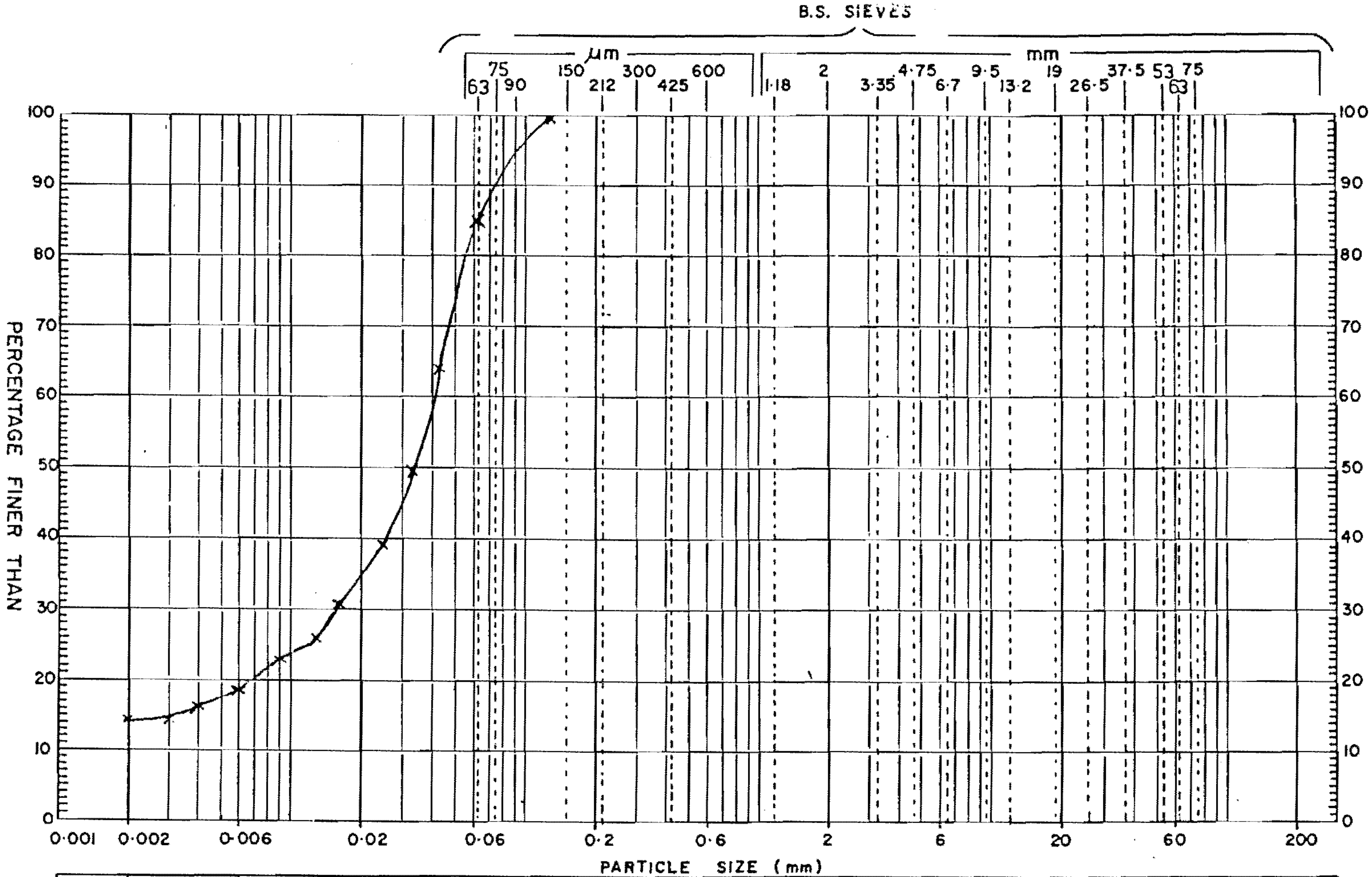



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: Undisturbed | SAMPLE No. W621 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.4

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

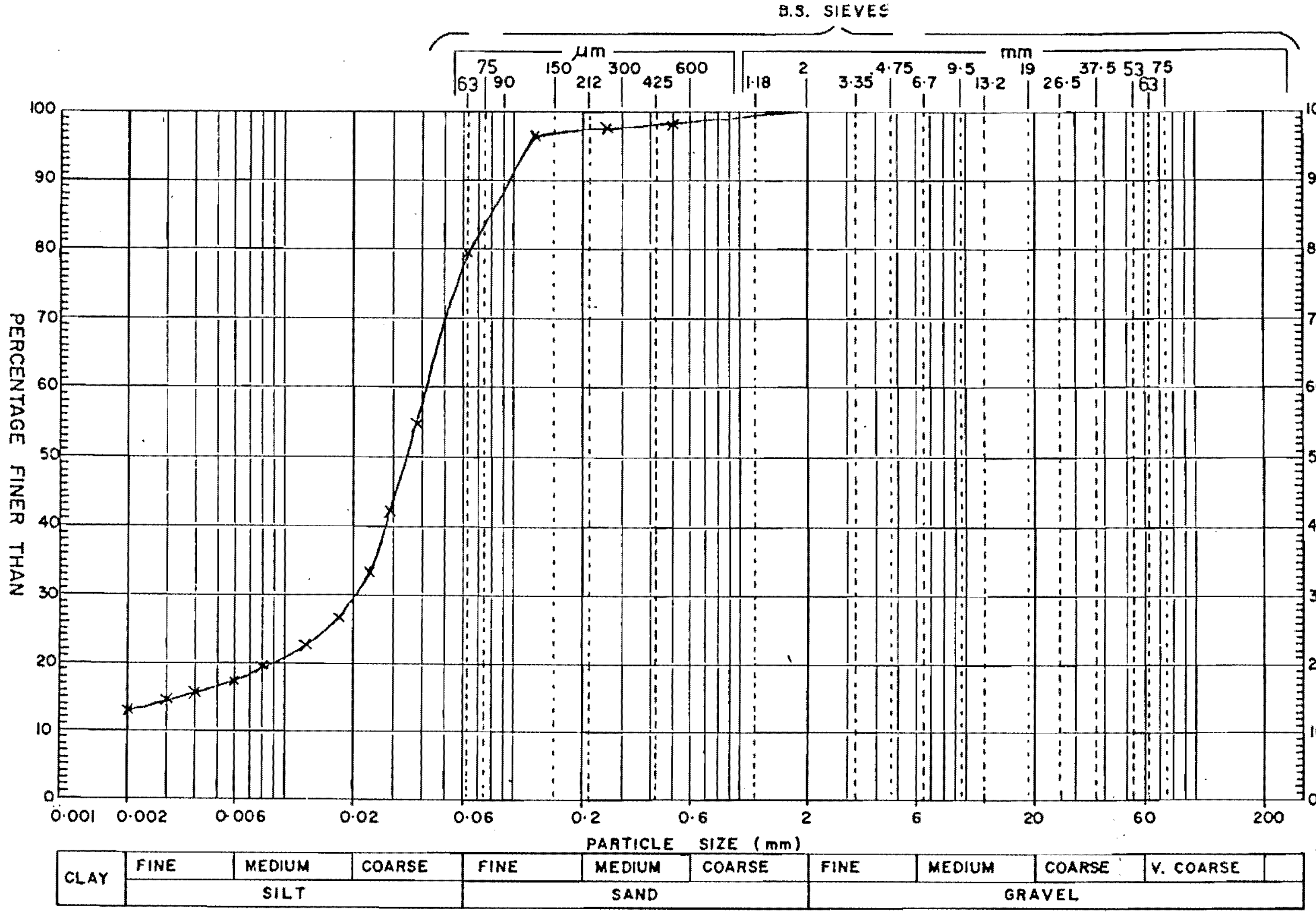



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: [illegible] | SAMPLE No. W423 | NATURAL / AIR DRIED / OVEN DRIED / UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.5

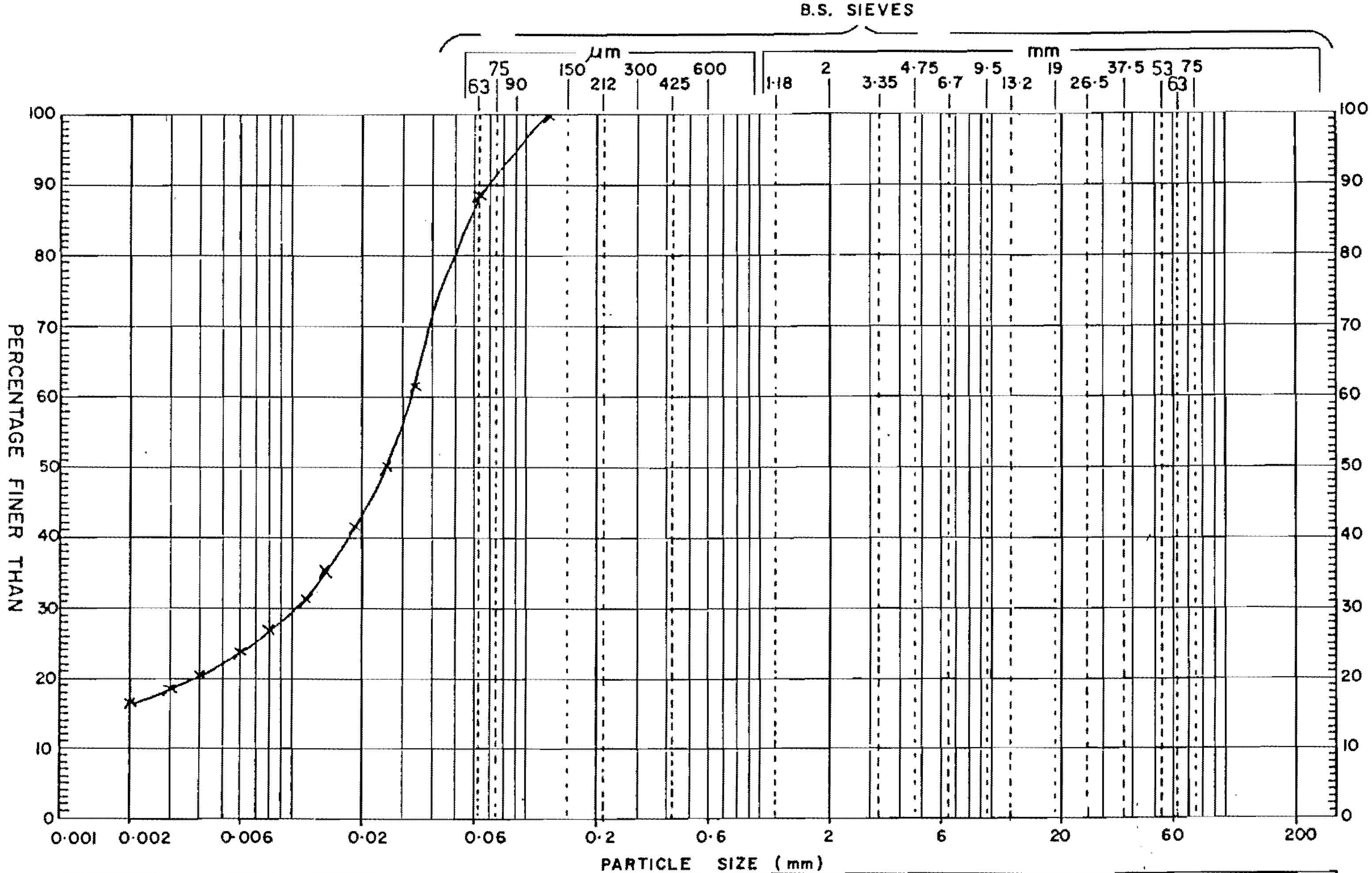
CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION




| | | | |
|---|---|---|---|
| JOB: Westmorland | SAMPLE No. W625 | NATURAL / AIR DRIED / OVEN DRIED / UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | \ | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.6

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

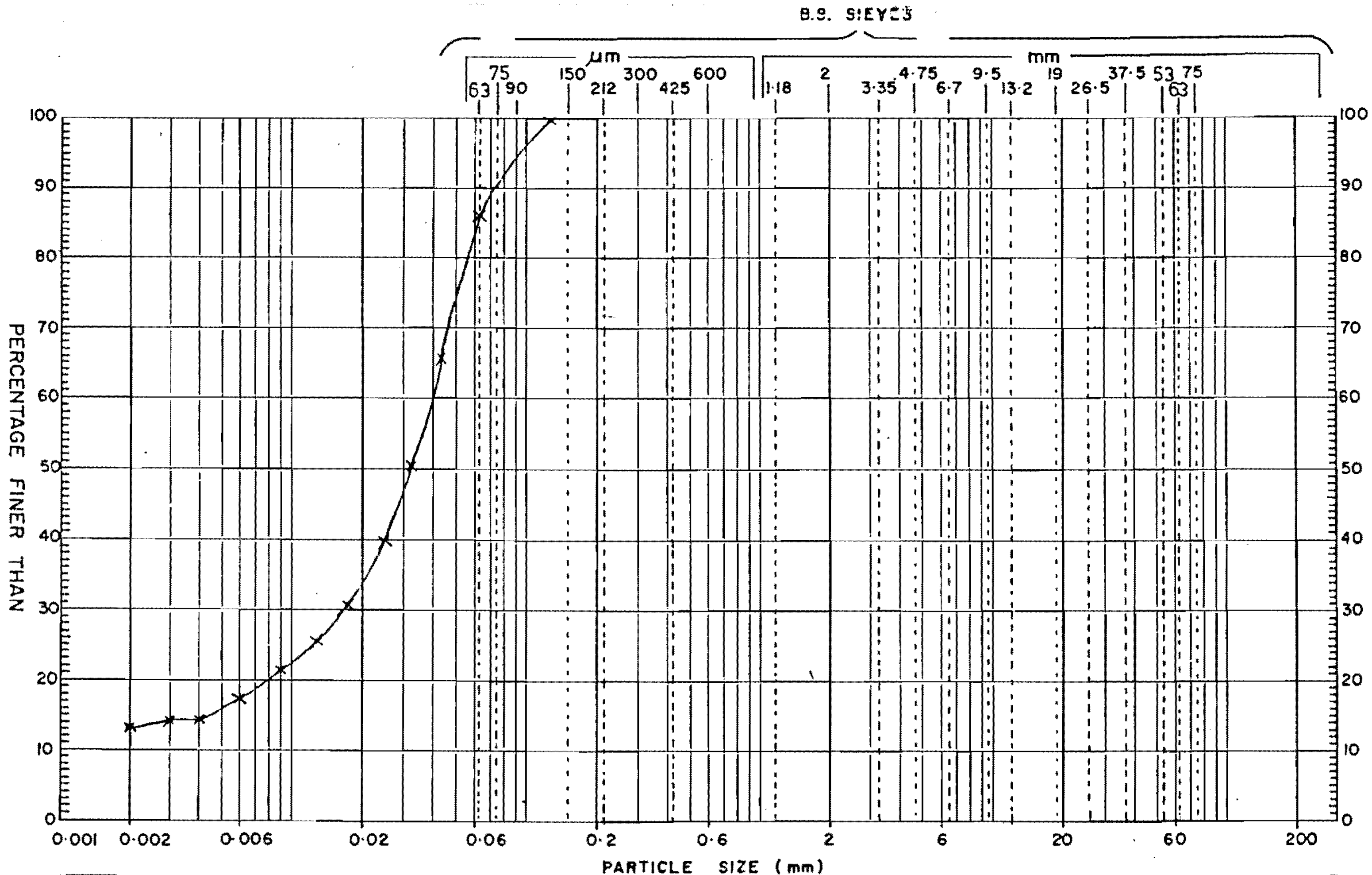



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: Waikanae (?) | SAMPLE No. W627 | NATURAL / AIR DRIED / OVEN DRIED / UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | ( | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.7

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

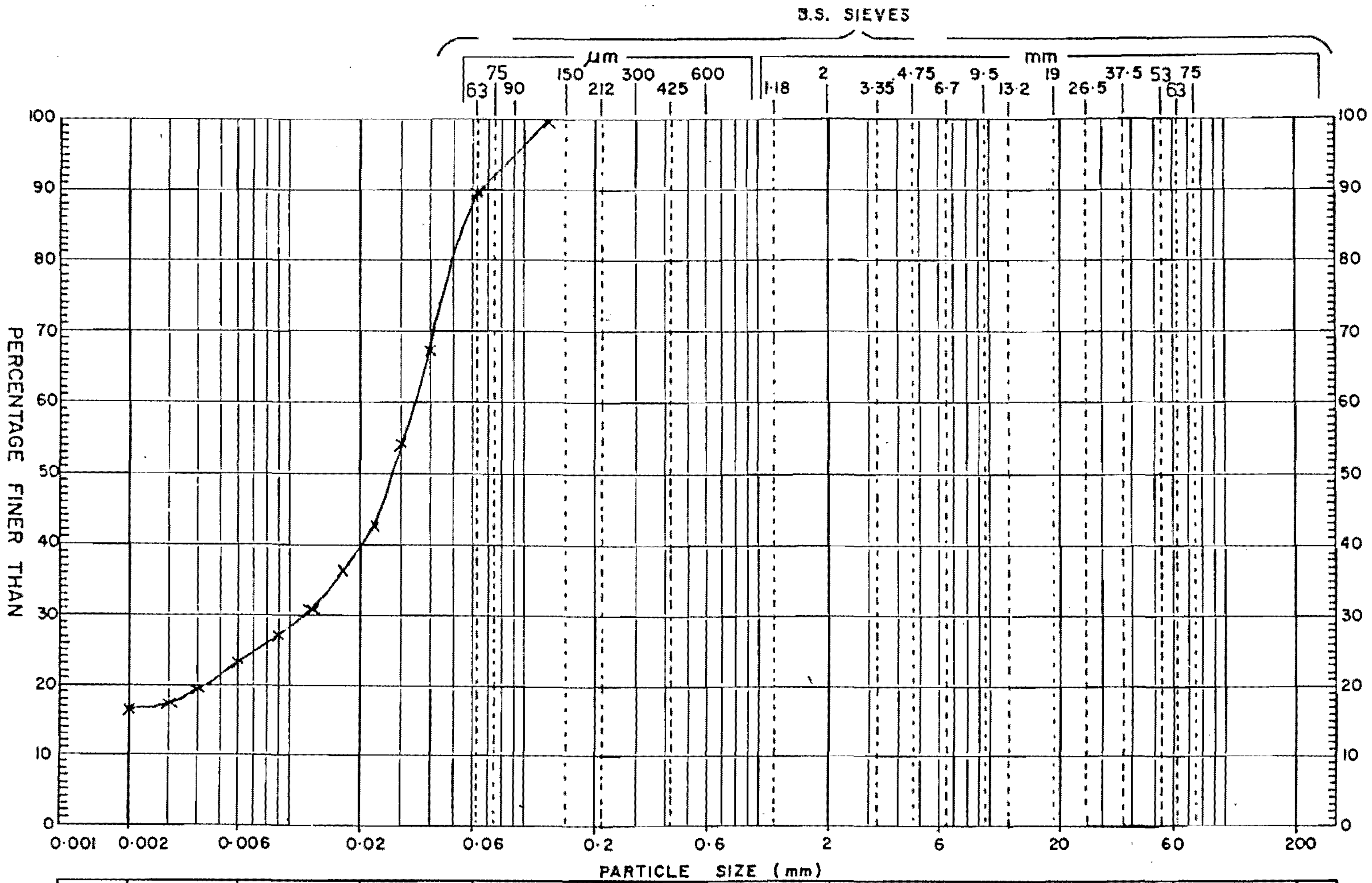



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: Westmorland | SAMPLE No. W630 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | \ | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.8

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

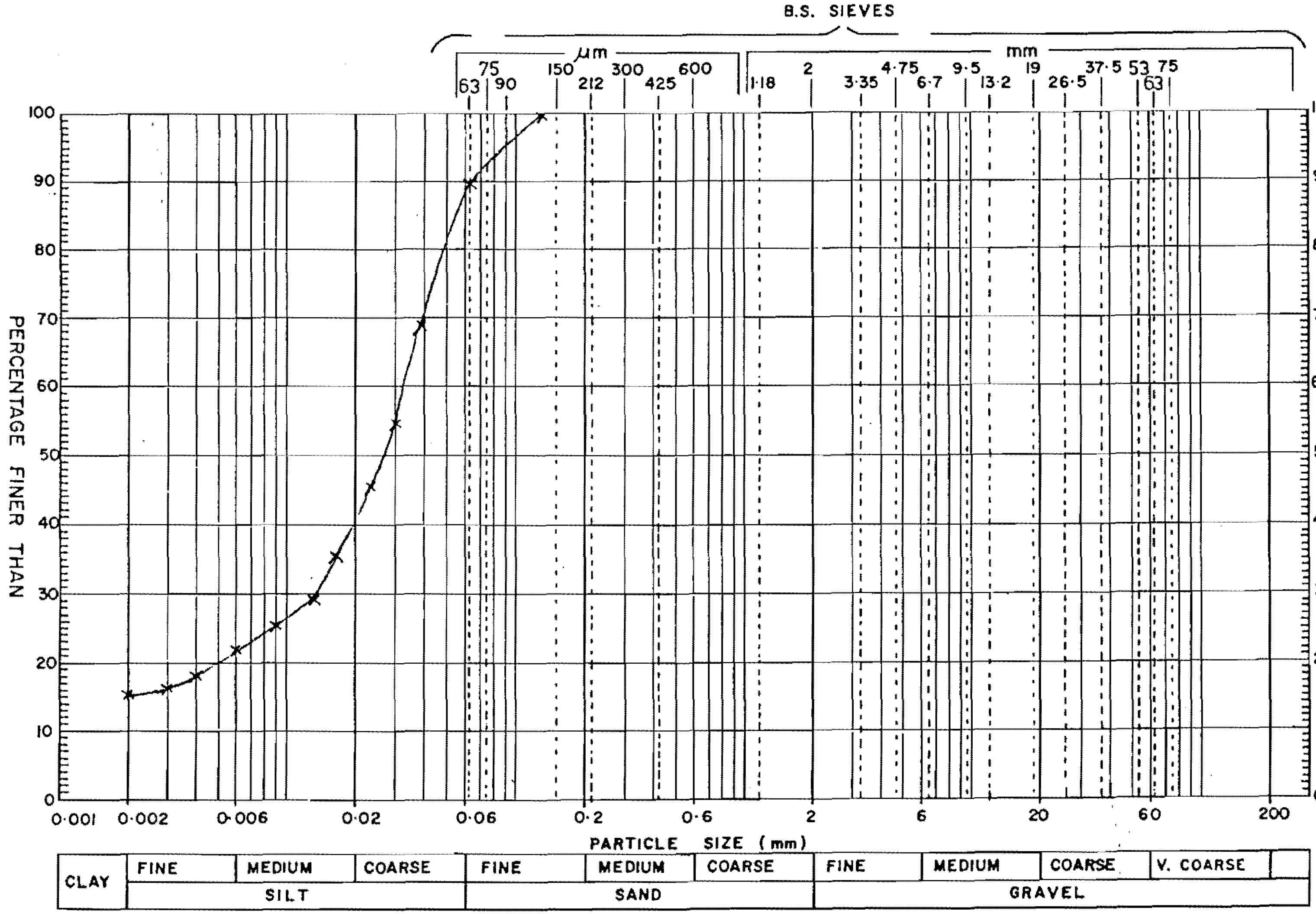



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: Westmorland | SAMPLE No. W431 | NATURAL / AIR DRIED / OVEN DRIED / UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.9

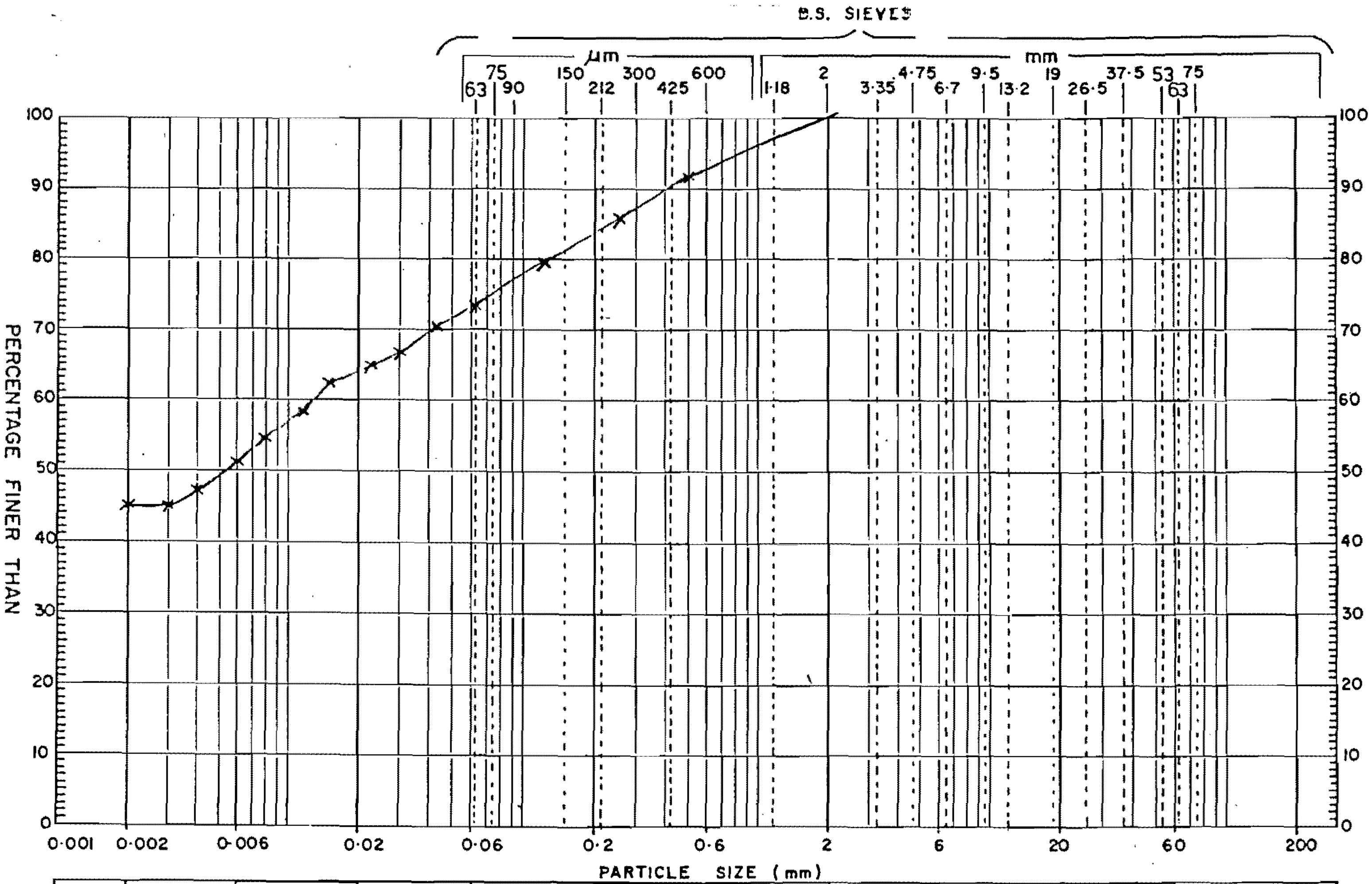
CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION




| | | | |
|---|---|---|---|
| JOB: Westmorland | SAMPLE No. W632 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.10

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

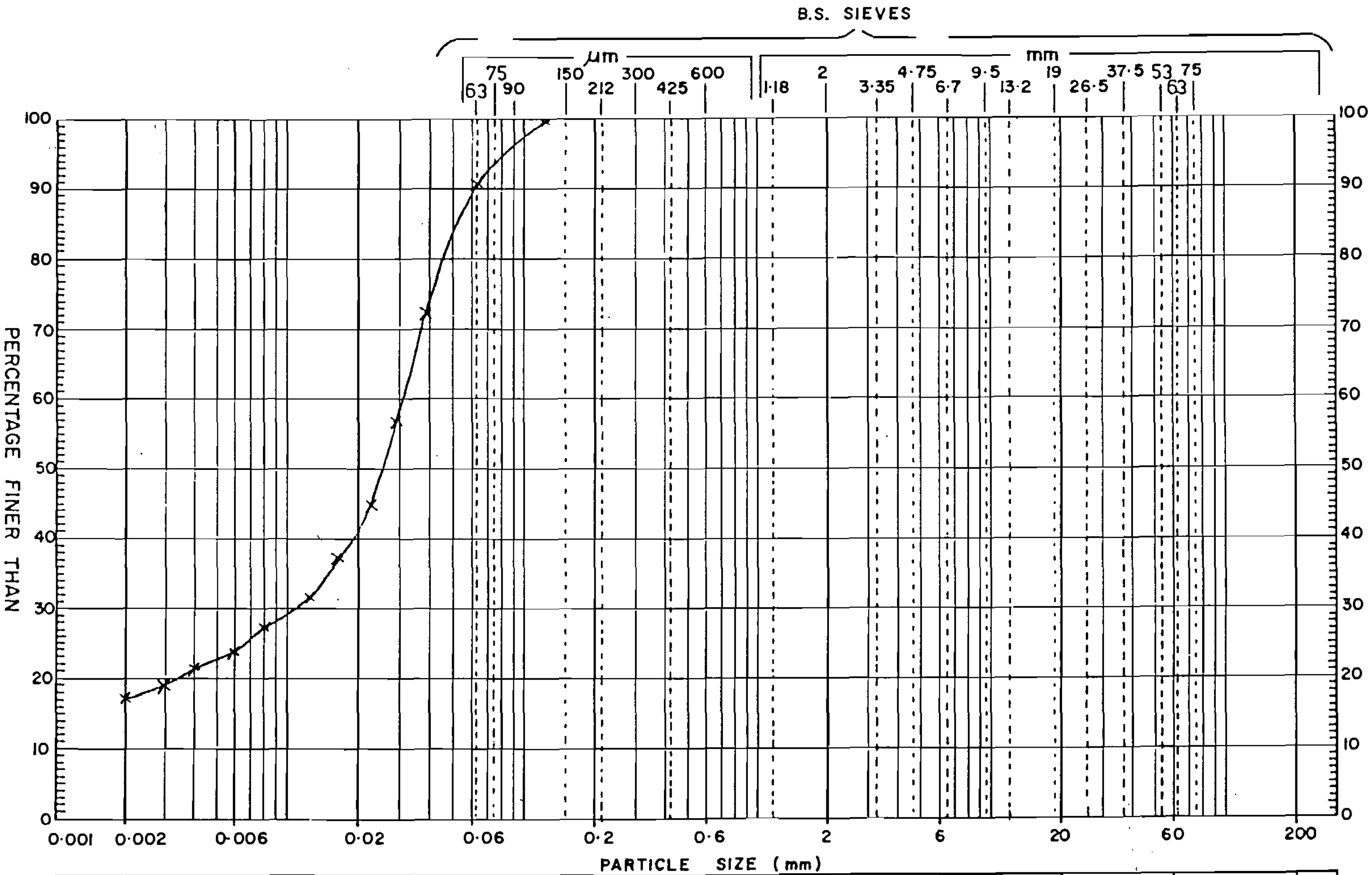



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| JOB: [illegible] | SAMPLE No. W633 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
|---|---|---|---|
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.11

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

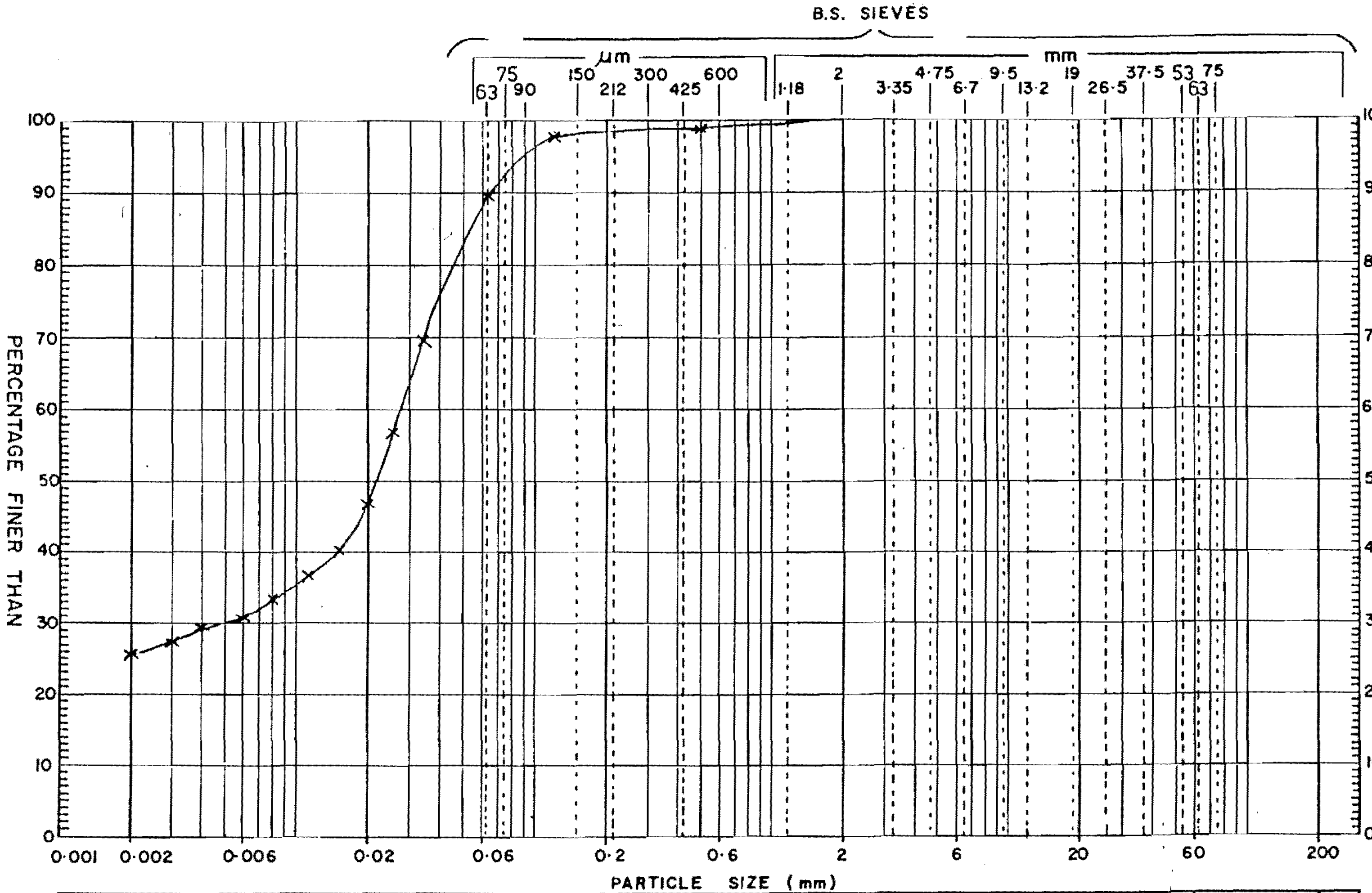



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | | |

| | | | |
|---|---|---|---|
| JOB: Westmorland | SAMPLE No. C1 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | \ | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.12

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION



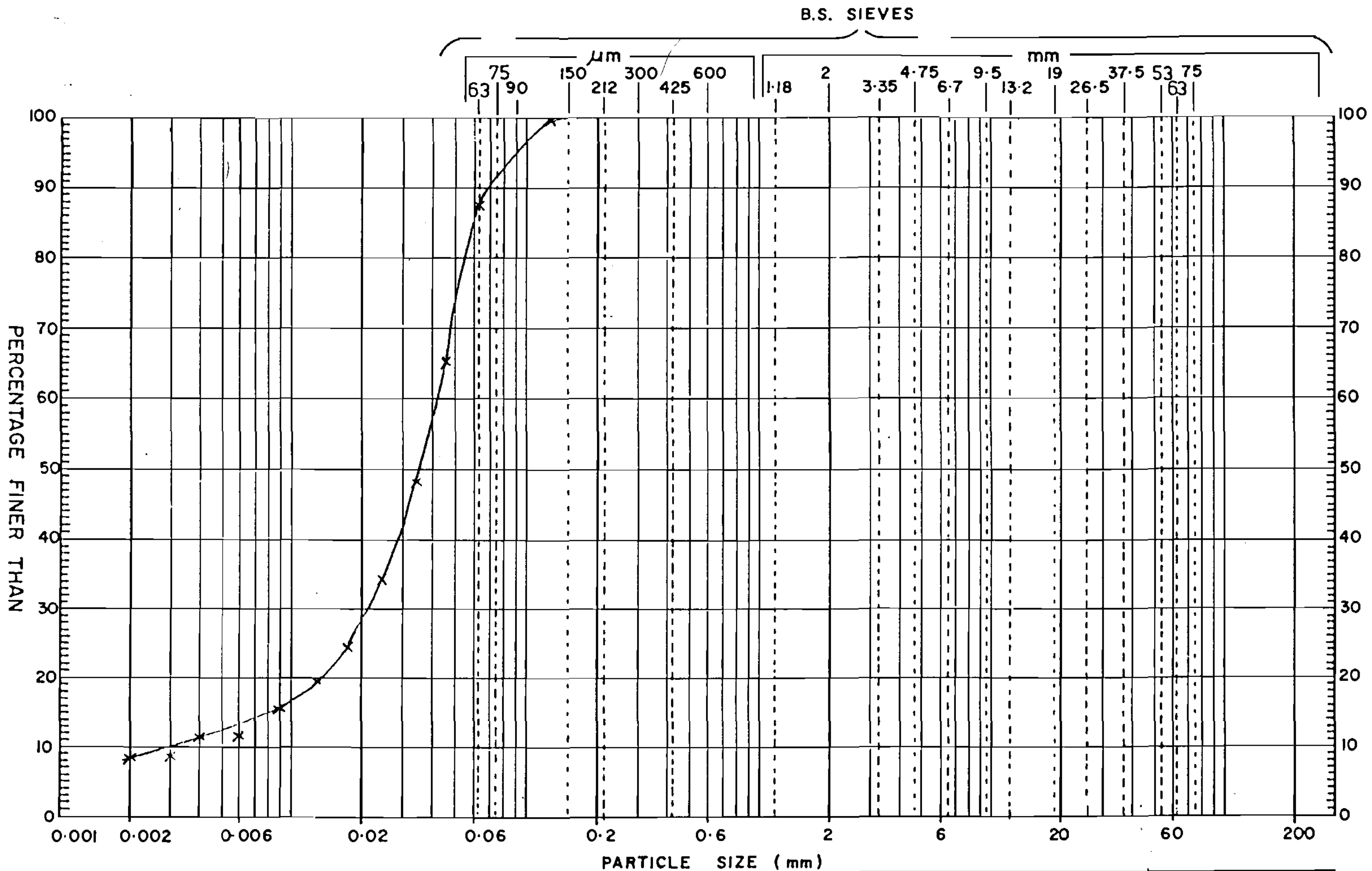

| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: [illegible] | SAMPLE No. C2 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.13

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

Test 9 (A)

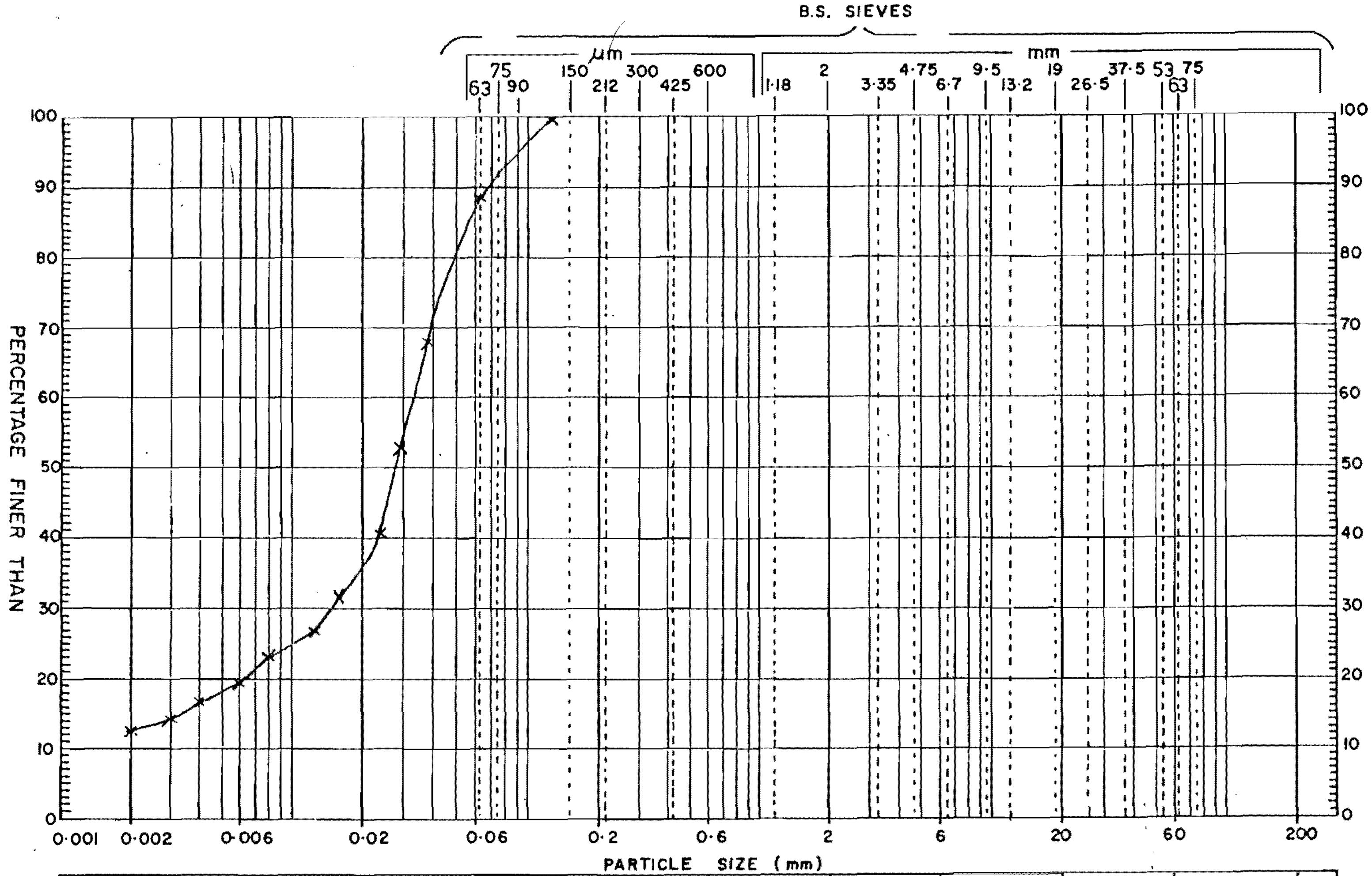



| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | |

| | | | |
|---|---|---|---|
| JOB: Ostrorland | SAMPLE No. C3 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | ( | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.14

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION




| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | | |

| | | | |
|---|---|---|---|
| JOB: [illegible] | SAMPLE No. C4 | NATURAL / AIR DRIED / OVEN DRIED / UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.15

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

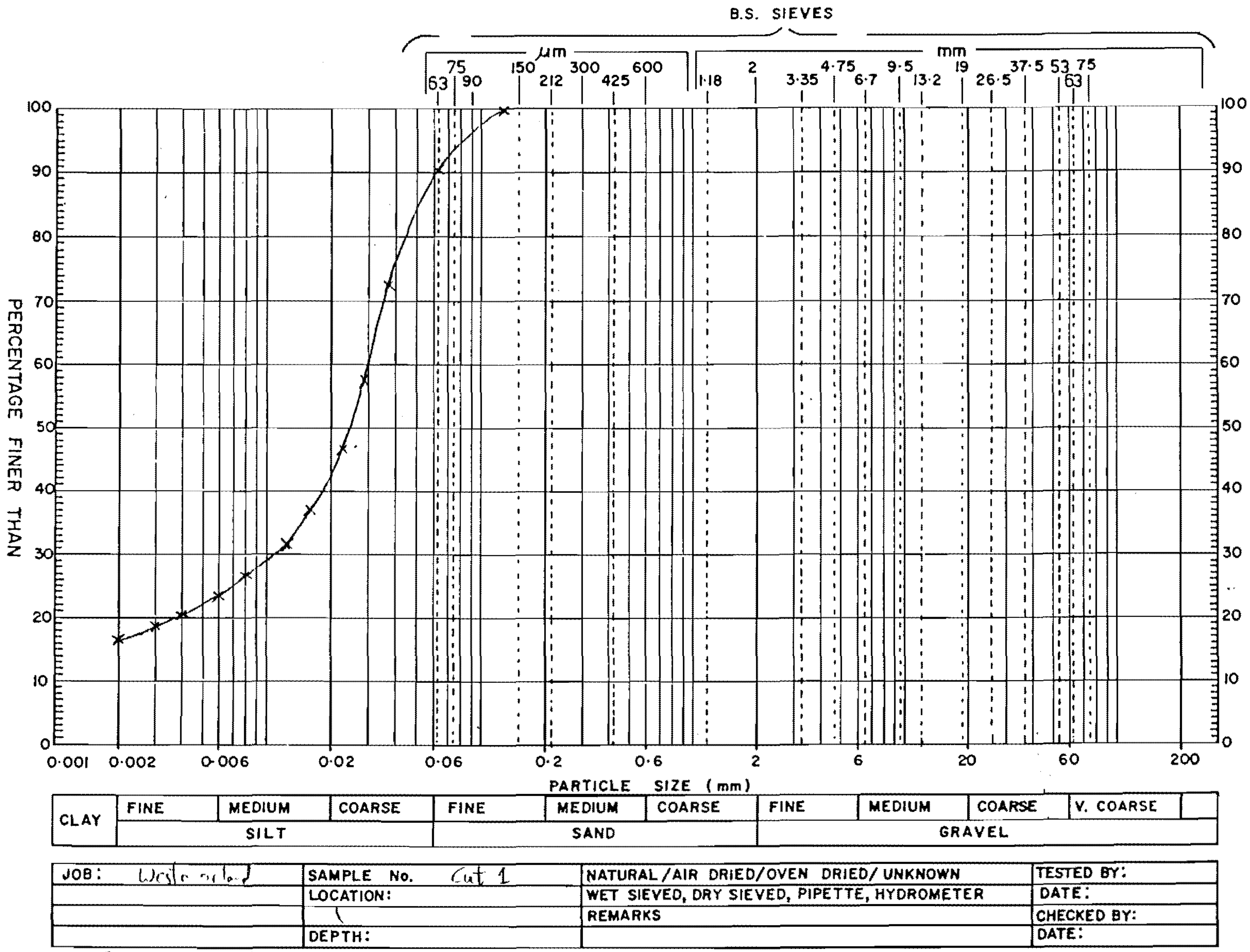
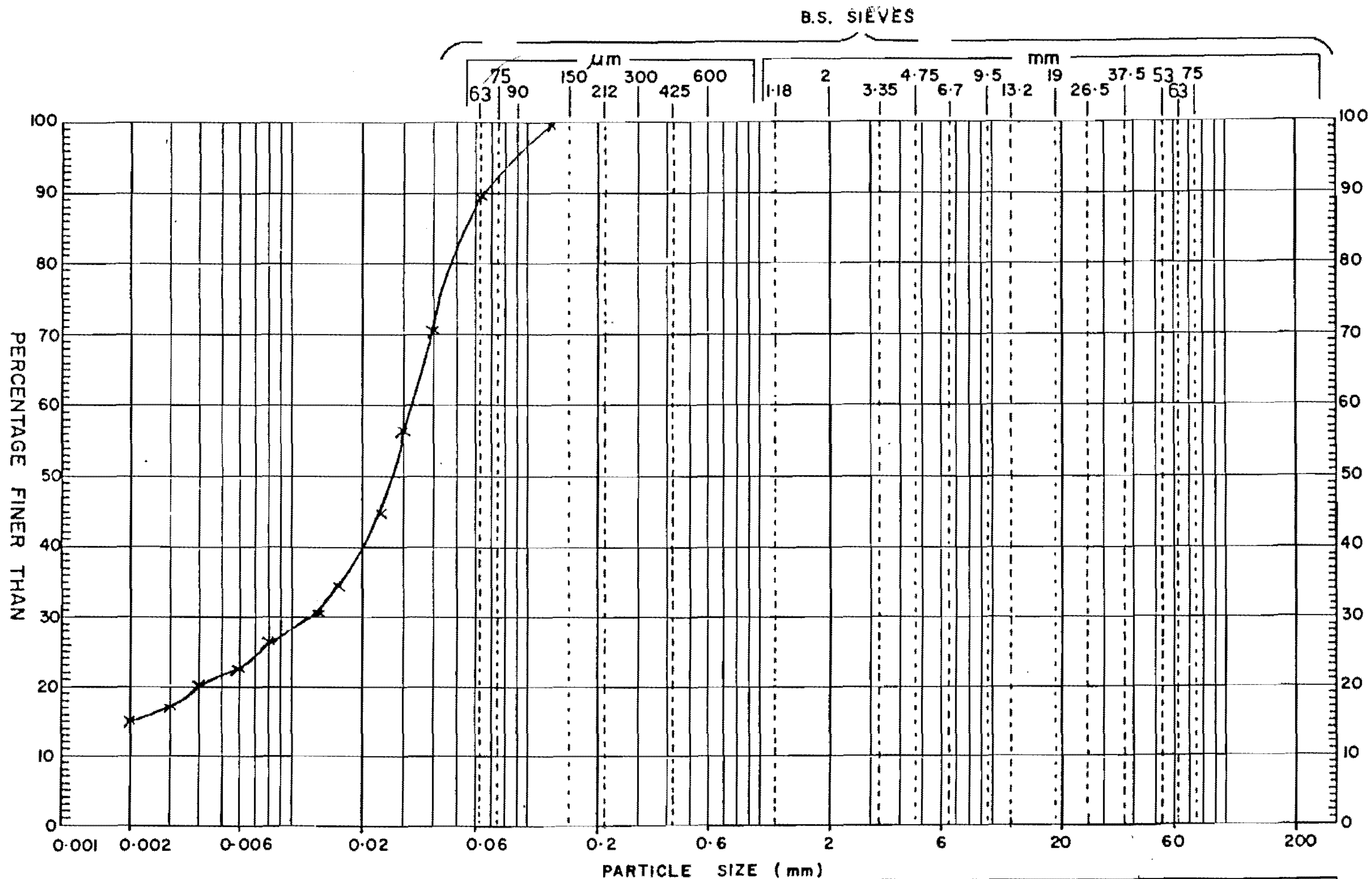



Figure A7.2.16

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION

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| CLAY | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | FINE | MEDIUM | COARSE | V. COARSE | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | SILT | | | SAND | | | GRAVEL | | | | |

| | | | |
|---|---|---|---|
| JOB: [illegible] | SAMPLE No. Fill 1 | NATURAL/AIR DRIED/OVEN DRIED/UNKNOWN | TESTED BY: |
| | LOCATION: | WET SIEVED, DRY SIEVED, PIPETTE, HYDROMETER | DATE: |
| | | REMARKS | CHECKED BY: |
| | DEPTH: | | DATE: |

Figure A7.2.17

CHART FOR RECORDING PARTICLE SIZE DISTRIBUTION